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Testing & Analysis of Short Span Wood Decks

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Abstract: The objective of this investigation was to provide additional data to support the current proposals regarding load distributions methods for short span wood decks.

Through the field testing of a number of different wood deck configurations, the current methods for determining design distributions, through a range of parameters, was performed.

The results of this investigation confirmed the validity of the current methods used in design for all the deck types listed except for an off-highway design used primarily by the Ministry of Natural Resources (MNR). The latter deck configuration exhibited a better distribution than is currently used by MNR in design.

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Table of Contents

1/ Introduction 1

2/ Site Descriptions 2

2.1/ Chats Falls Access Bridges, Ontario Hydro, Ottawa 2

2.1.1/ Victoria Island I Bridge 2

2.1.2/ Victoria Island II Bridge 3

2.1.3/ Ragged Chutes Bridges 3

2.2/ Witchdoctor Lake Road Bridge, MNR, Thessalon 3

2.3/ Mississagi River Bridge, MNR, Thessalon 3

2.4/ Henderson Park Bridge, MTO #10-113, Oakville 4

2.5/ Summary 4

3/ Instrumentation and Testing 5

3.1/ Punching Shear 5

3.2/ Witchdoctor Lake Road Bridge 5

3.3/ Mississagi River Bridge 6

4/ Test Results 8

4.1/ Prestressed Wood Decks 8

4.1.1/ Witchdoctor Lake Road Bridge 8

4.1.2/ Victoria Island I Bridge 8

4.1.3/ Victoria Island II Bridge, End Spans 9

4.1.4/ Victoria Island II Bridge, Main Span 9

4.1.5/ Ragged Chutes Bridges 10

4.1.6/ Summary 10

4.2/ Wood Sleeper Deck, Mississagi River Bridge 10

4.3/ Nailed Laminated Wood Deck, Henderson Park Bridge 12

4.4/ Summary 12

5/ Interpretation of Test Data 13

5.1/ Wheel Load Distribution 13

5.2/ Prestressed Wood Decks 13

5.2.1/ Deck Span 14

5.2.2/ Design Distribution 14

5.2.3/ Comparison of Experimental and Proposed Design Distributions 15

5.3/ Nailed Laminated, Henderson Park Bridge 15

5.4/ Wood Sleeper Deck, Mississagi River Bridge 16

Table of Contents

5.5/ Summary 16
6/ Conclusions and Recommendations 17
6.1/ Prestressed Wood Decks 17
6.2/ Nailed Laminated Wood Decks 18
6.3/ Wood Sleeper With Plank Decking 18
7/ References 19

Tables

1/ Details of the prestressed wood decks 21
2/ Vehicle loading for Mississagi River Bridge 22
3/ Distribution widths derived from measured deflections 22
4/ Proposed design distribution widths D_t for moment 23
5/ Assumed vs. actual edge beam stiffness 23
6/ Experimental distributions, Henderson Park Bridge 23

Figures 24

1/ Introduction

The Ministry of Transportation of Ontario (MTO) has been actively engaged in the development of wood bridges for more than a decade. The primary interest has been directed toward laminated wood decks because they have the greatest economic potential. These decks utilize the most readily available sizes of wood obtainable from current forests in Ontario.

Initially, in the mid 1970's, most of the attention was given to the longitudinal decks with spans over 3.5 m which were showing signs of mechanical deterioration [1]. It was through this work that the prestressed wood concept [2] was developed.

Over the last 5 years, more attention has been given to transverse wood decks [3] supported on girders with spans under 3 m. Although these decks have not demonstrated any serious problems, there is little test information available to validate the current design procedures, particularly for prestressed wood decks. This report presents the results of the field testing of seven bridges in Ontario which utilize transverse wood decks. Of the seven bridges, one has a nailed laminated deck, another has a wood sleeper-type deck and the remaining bridges have prestressed wood decks. The results are presented in several forms, with particular attention paid to the distribution of loads as compared to those used in design. Except in the case of the sleeper-type construction, all the results supported the load distributions currently used in design. In the case of the sleeper-type construction, which is used primarily by the Ministry of Natural Resources (MNR) on forest access roads, the results indicated that the load distribution was a little better than the MNR currently assumes in design.

2/ Site Descriptions

A description of each of the bridges is given below providing relevant dimensions and, where necessary, background information.

2.1/ Chats Falls Access Bridges, Ontario Hydro, Ottawa

Figure 1 shows a typical view of one of several bridges on an access road owned by Ontario Hydro, which leads into their Chats Falls Generating Facility on the Ottawa River, just west of Ottawa. Access to the facility is provided by four structures originally constructed in the 1920's for railway service. Subsequently, these bridges were converted for vehicular traffic use. These steel girder bridges supported creosoted railway ties until 1984, as shown in Figure 2, at which time the ties were replaced with prestressed wood decks [4]. This exchange represented the second time that the prestressed wood concept had been applied as a transverse wood deck.

Specific information on each of the Hydro Bridges is given in the subsequent subsections; however, the following items are common to each of the bridges.

- The wood decks are composed of Jack Pine No. 2 and better, pressure treated with Pentachlorophenol and oil.
- All wood deck panels were constructed and stressed off-site, ready for quick installation.
- The wearing surface is asphalt, crowned to provide drainage with a depth variance from 75 mm at centerline, down to 40 mm at the curbs.
- Tie-down of the decks to the steel girders is provided by pairs of 19 mm bolts inserted at 1524 mm spacing on each side of the girders with 150 x 300 x 13 mm washer plates countersunk into the wood surface.

2.1.1/ Victoria Island I Bridge

The typical cross-section of this bridge is given in Figure 3. The two heavy steel girders identify the locations above which the old rail lines used to be. These girders now support a 235 mm deep, prestressed wood deck, composed of 38 x 235 mm laminations. Although this structure is a single 30.5 m span, the wood deck is composed of 5 panels at about 6.15 m in length, joined together with flex-seal joints at the prestress bulkheads (Figure 4).

2.1.2/ Victoria Island II Bridge

This structure has two different compositions as depicted in Figures 5 and 6. The end-spans consist of triple steel girders each supporting a 184 mm deep deck, which runs the full length of the 9.14 m span. The main span has two large welded plate girders supporting a 286 mm deep deck composed of 5 panels each about 6.1 m long over the 30.5 m span.

2.1.3/ Ragged Chutes Bridges

The Ragged Chutes Bridges as shown in Figure 7 provide access along the face of the dam at the generating facility. Supported on concrete buttresses built out from the dam, the structure is composed of thirty-two 7.32 m spans of twin double girders as shown in Figure 8. Each span supports a single prestressed wood deck panel which is 184 mm deep.

2.2/ Witchdoctor Lake Road Bridge, MNR, Thessalon

The Witchdoctor Lake Road Bridge is one of two MNR bridges that have utilized a transverse laminated prestressed wood deck. Both are located on MNR logging access roads north of Thessalon, off Hwy 129. Built in 1985, the Witchdoctor Bridge presented the easiest access for testing purposes. Figure 9 provides a cross-section of the 15 m span bridge which consists of twin WWF 1000 x 244 girders supporting a 235 mm deep prestressed wood deck. The wood deck was constructed in-place over the full length of the bridge and was then stressed before being tied down to the girders. The wood is Jack Pine No. 2 and better, pressure treated with creosote. Unlike the Hydro bridges, but typical for MNR bridges, the deck has a wood plank running surface. The planks are 76 x 203 mm Jack Pine, grade unknown and not pressure treated. The deck is secured by 20 mm diameter bolts inserted at 600 mm spacing along the outside flanges of the two girders. The tie-downs are attached to steel channels as shown in Figure 9 which run the full length of the deck. The span is supported on rock filled pressure treated wood cribs, typical of many MNR access road bridges.

2.3/ Mississagi River Bridge, MNR, Thessalon

The Mississagi River Bridge is a standard MNR construction (shown in Figure 10), frequently used for logging access roads. The structure was tested to provide MNR support data for the design of the wood sleepers.

The bridge, built in 1983, has three spans, two measuring 21.4 m and one main span measuring 30.4 m. Figure 10 shows the cross-section of the east end span which was tested. The four WWF 1100 x

345 girders support a deck of 184 x 184 mm wood members, which are placed side by side and bolted to the girder flanges with 22 mm diameter bolts. These wood members are Jack Pine No. 1 pressure treated with Chromated Copper Arsenate. The wearing surface consists of roughly sawn 76 x 203 mm planks which are ungraded and untreated.

2.4/ Henderson Park Bridge, MTO #10-113, Oakville

The Henderson Park Bridge was built in 1981 and load tested as part of the MTO testing program in 1982 [3]. The current testing in 1988 was performed to detect the deterioration, if any, of the performance of the nailed wood deck. Additional details of the bridge are given in Reference 3. The 235 mm deep nailed laminated wood deck is composed of No. 1 Douglas Fir, pressure treated with Pentachlorophenol in oil. The deck is supported on four WWF 840 x 176 girders spaced at 1.42 m. There is no wearing surface. Dimensional layout is given in Figure 11. The wood deck is attached to the steel girder flanges using standard galvanized nailing clips as specified in the OHBDC [5].

2.5/ Summary

Of the seven bridges tested, the Mississagi and the Henderson Park Bridges are the only two which do not have prestressed wood decks. The original aim of this particular testing program was to evaluate the responses of prestressed wood decks. Table 1 lists the relevant details of the prestressing system for the five prestressed deck bridges.

3/ Instrumentation and Testing

Both the instrumentation and the test procedure are reasonably simple and repetitive for the testing of decks; therefore, they will be presented together for each bridge.

3.1/ Concentrated Load

In most cases, the testing of any type of deck slabs for the effects of wheel loads, is performed using the MTO punching shear equipment (Figure 12). The hydraulic ram fitted beneath the Kenworth testing vehicle has a capacity of 445 kN. The load is applied through twin 250 x 250 mm patches as shown in Figure 13. The figure also shows the locations of the LVDT displacement measurement stations on the aluminum frame. The frame is used to provide reference points away from the application of loading. It enables a local deck deflection profile to be measured with respect to points which will not move significantly during loading.

This equipment was used to test the Ontario Hydro bridges and the Henderson Park Bridge. The line of transducers was placed parallel to the main girders. The locations of the load patches, for each of the bridges tested, are given in Figures 14 to 18. In each case, the loading was placed at the centerline of the bridge, midway between supporting girders.

The maximum applied loading for the Ontario Hydro bridges was always over 300 kN, which is more than twice the factored wheel load of 138 kN given by the OHBDC. The evaluation section of the OHBDC, however, specifies a performance factor for deck slabs of 0.5. After moving this factor from the resistance side of the equation to the load side, the required unfactored resistance then doubles to 272 kN. Therefore, the successful testing of the Ontario Hydro bridges, to a load in excess of 300 kN indicates that these decks can be used for unrestricted highway loads.

The Henderson Park Bridge was not proof tested. The same loadings were applied as were used in the test conducted in 1982 as a means of evaluating its structural response only. The maximum load was about 200 kN which is still a high enough loading when compared to the factored design load.

3.2/ Witchdoctor Lake Road Bridge

The Witchdoctor Bridge with a running plank surface created problems for the use of the instrumentation frame. The latter provides instrumentation measuring deck displacements from the top of the deck. This is effective in the case of an asphalt surface or a bare concrete surface, as these surfaces conform to the same profile as the deck itself. However, for a plank surface, which is just nailed down, there is a tendency for the planks to be loose and undergo deformations which are different than those of the wood deck below. Therefore, in order to measure the actual movement of the wood deck on the Witchdoctor Bridge, the deflections were measured from the underside of the bridge.

Fourteen gauges were placed in a single row along centerline as shown in Figure 19. They were supported on wood planks which in turn rested on the bottom flanges of the steel girders. Placement on girders eliminated the need to adjust the measured displacements for the effects of overall bridge deflections as would be the case if the gauges had been supported by the ground below the span.

Loading was applied as before using the hydraulic ram on the MTO testing vehicle. The load was placed in 5 different positions along the bridge (Figure 19). In each case, the patch was located midway between girder supports directly over the line of instrumentation. As with the Hydro bridges, the maximum applied loading was over 300 kN.

3.3/ Mississagi River Bridge

The Mississagi River Bridge required the same instrumentation set-up as the Witchdoctor Lake Road Bridge. This bridge has four girders, as shown in Figure 10, but the running planks are only between the outside girders under the wheel paths of the traffic. The objective of the testing was to evaluate the load distribution of the wood deck system which would include the running planks. Therefore, testing had to be performed on the deck between the outside girders.

Twelve displacement transducers were placed in a row to monitor deflections of twelve individual sleepers (Figure 20). Two sets were used to monitor the same wood members between the outside girders on both the north and south sides of the bridge.

Loading could not be applied using the hydraulic ram, as the vehicle could not be moved eccentric enough to place the ram over the deck between the outside girders. It was necessary to use the wheels of the test vehicle itself, as shown in Figure 21. The two eccentric positions of the vehicle over the

width of the deck placed a line of wheels of the truck between the outside girders. The positions of the truck axles along the bridge deck are shown in Figure 20. The load levels of the vehicle and the forces induced by the axles on the deck are given in Table 2. The maximum loading is 90.5 kN per wheel, which is only 65% of the OHBDC factored design load. This deck was tested for behaviour only.

4/ Test Results

None of the decks displayed any signs of distress during testing with the exception of two instances involving minor laminate slippage in a prestressed wood deck. These two occurrences were instigated using very high loads as will be discussed in Section 5.

4.1/ Prestressed Wood Decks

The hydraulic ram was used to slowly increase the load so that a number of readings could be taken during the loading. This enabled the construction of deck deflection profiles with increasing load.

4.1.1/ Witchdoctor Lake Road Bridge

Figures 22 to 24 present several of the deck deflection profiles for the Witchdoctor Lake Road Bridge under three levels of load for three load positions. They are typical of the test results, indicating a reasonably consistent response for all test positions. Figure 25 presents the maximum displacement point for each load position plotted against an increasing load. Using a straight edge a slight curve to the right of all lines can be detected, indicating a slightly non-linear performance becoming more pronounced at higher load levels. This behaviour reflects the properties of wood which are known to be non-linear [6] in the same fashion.

4.1.2/ Victoria Island I Bridge

Two typical deflection profiles for the Victoria Island I Bridge are given in Figures 26 and 27. Figure 26 represents a load applied to the deck away from the stiff anchorage bulkheads near the ends of the deck panels. The other demonstrates the effects of the stiff steel bulkhead on the deflection profile of the deck when the load is placed near the end of the deck panel. The loading head is free to rotate in the direction of the instrumentation, and this is apparently the case in Figure 27, due to the very stiff edge condition created by the steel channel at the ends of the deck panels.

Figure 28 provides a number of load-deflection comparisons which again display a slightly non-linear performance with several additional features. All curves display a lower stiffness with loads below 50 kN. This feature is attributed to the neoprene spacer which was between the deck and the tops of the steel girders (Figure 3). This spacer was used to provide a more uniform bearing, as the girders have steel rivet heads protruding from the top. The line for position 5 demonstrates a stiffer performance which was common to the edge panel loadings. This stiffness is believed to be due to the effects of the steel bulkhead which was adjacent to this load position. However, in conflict with this is the line for position 8, which was also adjacent to a bulkhead and shows no increased stiffness. There does

not appear to be a ready explanation for this discrepancy other than the possibility that the wood in the deck at that location was made up of particularly low modulus material. These same two positions were used to evaluate the effects of increasing the load as high as possible in an attempt to create some laminate slip. This was successful as shown by the abrupt increases in deflection at the highest load level for positions 5 and 8. Although the loads reached as high as 400 kN before the slip occurred, the data was only recorded after slippage. Such slippage was not observed at any of the load positions away from the bulkheads. The stiff boundary created by the bulkheads resulted in high interlaminate shear stresses. It should be noted however that a 400 kN wheel load is nearly three times the factored OHBDC design load (including dynamic load allowance).

4.1.3/ Victoria Island II Bridge, End Spans

The end spans of the Victoria Island II Bridge appeared to demonstrate consistent performances as shown in the deck deflection profiles given in Figures 29 and 30. However, the load-deflection curves in Figure 31 are not as consistent. These lines indicate that stiffening occurred with an increased load, except in the case of position 7.

If a closer look is taken at this phenomenon, it becomes evident that the stiffening occurred primarily at the beginning of loading. This stiffening was probably due to the fact that the deck was not in proper contact with the supporting girders. Figure 5 shows that there were three girders for each support line, creating a wide bearing area for the wood deck. Under this arrangement, it is unlikely that a consistent uniform contact between the deck and the girders will be achieved. This will affect the measured response of the deck, particularly at lower loads, until the deck is forced into full contact with the girders in the vicinity of the loading. As can be seen in Figure 31 at loads of over 100 kN, these lines again curve slightly to the right.

4.1.4/ Victoria Island II Bridge, Main Span

Figures 32 through 34 display some of the test results for the main span deck of the Victoria Island II Bridge. These plots indicate that this deck performed reasonably consistently with those of the previous discussions. The load-deflections given in Figure 34 display a higher stiffness for the edge loading and some initial stiffening as load is applied for positions 1 and 3. However, there appears to be some continued stiffening action for positions 1 and 2. These two points were directly beside each other on the same panel and therefore it appears this phenomenon is repeatable. This continued stiffening is in contrast to the previous responses and no definitive explanation can be offered at this time.

4.1.5/ Ragged Chutes Bridges

Figures 35 and 36 display some representative deck deflection profiles from the testing of the Ragged Chutes Bridges decks. Figures 37 and 38 give a number of load-deflection plots which show all of the previously discussed phenomena.

4.1.6/ Summary

The results of the punching tests of the prestressed wood decks show that the decks appear to exhibit a reasonably consistent performance. There are a number of observations which may be summarized as follows:

1. The deflection profiles indicate that most of the deck deformation, for loads away from the ends of the panels, appears to occur within 0.5 m on either side of the load regardless of the deck geometry tested. This was confirmed by plotting as much as 75% of all the test information that was collected.

2. The tests performed immediately adjacent to the steel bulkheads indicated that the bulkhead acted as a rigid support to the wood deck. In each case, the second gauge from the centre of the frame was within 25 mm of the bulkhead on the wood deck and registered zero deflection consistently for all practical purposes.

3. For each deck panel the flexural rigidity varied within a narrow range, exhibiting both a stiffening and softening action to a small degree.

4. Interlaminate shear strength of the decks was very high.

4.2/ Wood Sleeper Deck, Mississagi River Bridge

Figures 39 to 44 display some of the deck deflection profiles from the testing of the Mississagi River Bridge. The first two figures represent the action of a single axle where the responses conform to a deflection profile expected from a continuum. Therefore, the plank surface, which is the only connection between the individual wood sleepers, must provide a reasonable distribution of the wheel loads. There does not appear to be a significant increase in the maximum deflection between the wheel loading of 137 kN and 181 kN, particularly in Figure 39. This result may have been caused by the poor repeatability in vehicle placement when using the wheels as opposed to the hydraulic ram. The positioning in which it is most difficult to achieve accuracy is laterally, or positioning between the girders. Obviously, the response will be sensitive to this position considering the wheel

width is over 0.5 m while the clear distance between the girders is just over 0.9 m. This is one of the reasons why the hydraulic ram is used to test for local deck responses that are sensitive to small changes in the load position.

Figures 41 and 42 show the effects of two equally loaded axles. As mentioned before, it was very difficult to obtain accurate wheel positioning and in several cases the vehicle was removed and repositioned. Due to the heavy weight of the vehicle in load level 4 (i.e. 181 kN wheel loading) little effort was put into repositioning the wheels for accuracy. The figures demonstrate the effects of this where the heaviest loads do not conform to the same shape as the lesser loads.

The last two deck deflection profiles display an unexplainable phenomenon where the two axles are of equal loading but the maximum deflections under each wheel differ by nearly three times. This appears to be repeated in all three loadings and on both sides of the bridge. Details of the tie-down of the deck and attachments were reviewed to determine if something could be stiffening the members in this area. Nothing was found since the connection details including guide posts etc., are repeated every 1.055 m along the bridge. Therefore, any increased stiffness should also have been evident somewhere in Figures 39 and 40 and yet these deflection profiles appear to be smooth over at least 1.5 m.

One reasonable explanation, which would be consistent with the observations, is a probable variation in the material properties of the individual sleepers, specifically the modulus of elasticity. In a laminated wood deck, where there are a large number of of small members performing together, the effects of high variations in stiffness (coefficients of variation of 40 to 50 % are not uncommon) would be insignificant. In the case of the wood sleeper type construction, with the larger size independent members, one member with a relatively high stiffness (possibly even a different species) could have a significant affect on the displacement performance. If the member directly under axle no. 2, for the positions displayed in Figures 43 and 44, was considerably stiffer than the rest, then observed performance could be justified. Note also that, since the sleepers extend across the full width of the deck, the same would be observed on both sides of the deck which is also consistent with the figures.

4.3/ Nailed Laminated Wood Deck, Henderson Park Bridge

The deck deflection profiles from some of the testing of the Henderson Park Bridge are given in Figures 45 to 47. These appear to be typical of the measured results and a visual comparison with the

responses measured 6 years ago [3] indicates that there are no dramatic differences. The changes in distribution calculated from these results will be presented and discussed in Section 5.

Figure 48 presents the load-displacement relationship for each of the test positions using the deflection at the load point. These results demonstrate the same performance observed 6 years ago with a stiffening action during the initial loading. This action becomes more pronounced with age, as the nail holes in the laminates enlarge and a larger movement of the deck is necessary to engage the nails before the deck can effectively resist the load. Figure 49 displays the range of load-displacements (i.e. the upper and lower curve boundaries) taken from Figure 48 as compared to the same measured limits from the tests of 6 years ago. Although there is not a profound difference there is a definite tendency for the range to shift to the right, confirming the assumption of increased mechanical performance due to loosening of the nails.

It is necessary to note that the loosening of the nails in the laminates will be much more pronounced under the wheel paths. In contrast to this, the tests were performed along centerline where the hydraulic ram could easily be positioned. However, the actual wheels of the traffic will not reach the centerline, so any loosening action of the nails in that area will only be a secondary effect of the loadings in the adjacent deck spans.

4.4/ Summary

There is an abundance of test data from these tests with only a small amount presented here. The data presented was selected to show all the observed actions for each bridge deck.

5/ Interpretation of Test Data

In the subsequent discussions, the test data used in derivational calculations incorporates all the available data and is not limited to the information presented in the previous sections. In many cases, mean, low, and high values will be provided for each calculation.

5.1/ Wheel Load Distribution

Bakht [3] presents a technique for determining the distribution width for a wheel load using test deflections. The distribution width D_t is calculated with the following formula:

$$D_t = \frac{A}{w_{max}}$$

where A is the total area under the deflected shape and w_{max} is the peak value of deflection (Figure 50). The area under the deflected shape can be determined quite readily using numerical integration techniques such as Simpson's Rule. Distribution widths were determined for all the test data using this approach and the results are given in Table 3.

Reviewing the distribution values in Table 3, there appears to be little variation of the mean values under different load levels. At the same time, the results for the Witchdoctor and Victoria Island I Bridges have large standard deviations. Although the standard deviation has been calculated, it is important to note that the values are based on a small sample size. The results from the Witchdoctor Lake Bridge utilize only three of the five points tested, because the other two locations were near the edge of the deck. Difficulties with some of the data from the Ontario Hydro bridges resulted in only about 5 points being used for each deck type in determining the distributions for all but the Victoria Island I Bridge for which only 4 points were used.

5.2/ Prestressed Wood Decks

One of the primary objectives of this work, as discussed previously, was to generate data to support the design distributions for prestressed wood decks. Bakht [3] presents the proposed design distributions for short span prestressed wood decks and concludes that the distribution width D_t is independent of the deck thickness and depends on the girder spacing or effective deck span. As will be seen later, the assumption for the span of the deck is critical and can seriously affect the design.

5.2.1/ Deck Span

In the past, the range of design spans taken for decks supported on beams or girders has gone from the girder spacing (centre to centre of girders) down to the distance between points of inflection (zero moment) in the span of the deck. Figure 51 shows an example of a deck continuous over multiple girders with three possible span assumptions. On long spans the effect of the support width, (i.e. the flange width, is quite small). However, for these short span decks the support width should not be ignored as it will seriously affect the design . In addition, when considering the distance between points of inflection, the span should occupy about 68% of the girder spacing. It is important to consider both of these points because the design work performed by Bakht [3] is based on a simply supported span.

5.2.2/ Design Distribution

As discussed in detail by Bakht [3], the moment in a transverse deck on longitudinal girders is made up of two components. There is overall bending and local bending in the vicinity of a wheel. For decks with large spans, on girders which are very stiff compared to the decks, the overall bending is usually quite small compared to the bending between girders. The opposite is true for bridges with multiple girders or stringers which are less stiff and have small spacings, such as sawn wood stringer bridges, where the stringer spacing can be as low as 0.5 m. The range of spans for which the prestressed wood deck can and has been applied, takes it right through the transition of these two types.

Table 4 provides the proposed design width D_t for prestressed wood decks taken from Bakht [3], which covers the span range for the bridges tested and accounts for the applicable bending influences. For the eccentric or edge loaded decks, the stiffening provided by the bulkhead was taken as 0.2 SD_y, which is equal to a width of deck equivalent to 20% of the deck span. Note that, as given in Table 4, for the shorter spans, very little additional edge stiffening would be required to ensure that the edge loading did not govern the design. That is, the effective D_t for the edge loading would become larger than that for the centrally loaded case. Following through on this point, Table 5 provides the edge beam stiffness for each of the bridges tested, along with the value equivalent to 0.2 SD_y. In all cases, the actual stiffness is much greater than assumed. If the maximum deflection indicates maximum flexural stress in the deck, the deflection profiles from testing confirm that none of the edge loaded cases were more critical than the centrally loaded cases.

For the comparisons given in subsequent discussions, only the centrally loaded cases will be used.

5.2.3/ Comparison of Experimental and Proposed Design Distributions

Figures 52 to 56 display the experimental distributions as compared to the proposed design values from Table 4. The test results are presented as a range, covering a rectangular area. The upper and lower limits of the D_t values are the maximum and minimum experimental values at a loading, as close to the factored OHBDC design load as possible. The upper and lower bounds for the span are the clear distance between flanges and the points of inflection. In every case, as we move towards the assumption that the span is the distance between the points of inflection, the experiment results indicate that the proposed design distributions would be conservative. The points of inflection were based on the assumption that the tie-down of the deck, as shown in the cross-sections on each figure, would be taken as a reaction point when a load was placed at centerline, as demonstrated in Figure 57.

In the case of the Witchdoctor Bridge, the tie-downs along the outside flanges were nearly continuous over the length of the bridge, so to assume continuity would be quite reasonable (Figure 57). Therefore, the assumed span for the Witchdoctor Bridge should be very close to the distance between the points of inflection.

On the Ontario Hydro bridges the tie-downs were widely spaced, alternating at about 1.5 m spacing along both flanges of a girder(s). The effectiveness of this arrangement to induce deck continuity is questionable. However, some continuity must have been achieved. Figures 53 and 55 are the only two comparisons from the Ontario Hydro tests which have part of the experimental area below the design curve. Even so, it would take only a marginal decrease in the assumed span, below the clear span distance, to remain on the conservative side of the design curve.

Figure 58 summarizes the results from Figures 52 to 56 using only the average distribution values. Contrary to the initial impression that the distribution widths do not vary greatly, this figure demonstrates that a definite relationship exists with the span. The points representing the distance between points of inflection provide a trend nearly parallel to the predicted point or design curve.

5.3/ Nailed Laminated, Henderson Park Bridge

The experimental distributions from the Henderson Park Bridge are also given in Table 3. These values are based on 10 test locations; therefore, the standard deviation, which reflects reasonably good repeatability, can be taken with more confidence.

Table 6 provides the data from the tests performed in 1982 as compared to the same data from the current testing. There are two important things to note. The first and foremost is that both the mean

and minimum distribution values are slightly worse than when the bridge was only one year old. This confirms the belief that nails loosen with time thereby reducing the deck's capacity to distribute load. Secondly, the coefficient of variation of the latest test results is smaller, indicating that the repeatability improves after repeated loadings.

5.4/ Wood Sleeper Deck, Mississagi River Bridge

The distribution results from the Mississagi River Bridge are also given in Table 3 but these must be viewed with caution. Figures 39 to 44 show that the measured deflection profiles did not cover the full extent of the actual deck movement. Therefore some interpolation was necessary as indicated by the dotted lines in Figures 39 and 40. The area under the deflected shape was determined using this projection. The same was done using Figures 43 and 44 by isolating the effects of the two axles.

The design of these decks by MNR usually assumes that three wood members carry the design wheel, constituting a distribution width of about 0.6 m. Taking into account possible errors that could occur when interpolating the test results, the calculated distributions indicate an improvement over the design assumptions with a minimum D_t of 0.98 m. Four members carrying the wheel load would require a D_t of 0.81 m and five would need 1.01 m.

5.5/ Summary

Test results are always susceptible to variations due to poor vehicle placement, poor instrumentation set-up or faulty operation, as well as short-term response to thermal changes. However, the apparent repeatability of the performances of these decks is confirmation that no serious errors or factors influenced the tests to any great degree. Although the standard deviations should be viewed with some caution, the mean distributions of all but the Mississagi River Bridge are believed to be representative of the decks that were tested.

6/ Conclusions and Recommendations

6.1/ Prestressed Wood Decks

If the distributions presented in the previous section are considered, it appears that the design values proposed by Bakht [3] are conservatively acceptable.

However, in using these proposed design values it is imperative that the correct span assumption be utilized. Discussions with Bakht led to the recommendation that the clear distance be used in the calculation of the undistributed moment. Subsequently, the distribution width over which this moment is to be carried should be obtained assuming the span is equal to the distance between points of inflection for continuous decks.

It would also be wise to include some note in the design specifications to point out to the designer the effects of tying down these low stiffness decks over wide flanged girders. As was the case in each of these tests, the effective span was reduced. If the clear span were to be assumed, the resulting design distribution width may be too large in some cases.

Looking at Figure 58, it appears that the range of spans tested was quite small and the trend indicates that there could be some discrepancy at higher spans. However, quite a number of longitudinally laminated decks with spans over 4.5 m [1, 6] have been tested along with extensive laboratory testing, [2] all of which have confirmed the orthotropic response of these decks. The proposed design for the wider spaced girder bridges is based on orthotropic performance assuming the girders provide rigid supports. However, it is suggested that if a prestressed wood deck on girders spaced at between 3 to 4 m is ever constructed, a behaviour test would be valuable in projecting the results displayed in Figure 58.

The effect of edge stiffening due to the anchorage channels is quite significant. The stiffness of the channels used on these decks was significantly higher than the assumed 0.2S D_y and in each case this resulted in the centrally loaded condition being more critical. It is important to recognize that the channels on these bridge decks were selected from the OHBDC [5] and represented the minimum sizes. Therefore it appears that as long as the designer follows the OHBDC for the bulkhead design, he/she will only have to consider the centrally loaded case in the design of the deck for moments.

6.2/ Nailed Laminated Wood Decks

There is little that can be added to the previous discussions except by way of presenting Figure 59 which was taken from Bakht [3]. This figure displays the OHBDC design width for nailed decks as a horizontal line at 0.4 m. The figure also gives the test data which shows that D_t is actually a function of the deck thickness. However, as has been presented, D_t is also a function of the age of the deck. The actual D_t for a deck moves closer to the 0.4 m value with time as the deck delaminates. The test results from the Henderson Park bridge agree with this projection.

In order to accommodate for the continued deterioration of the deck, it is recommended that the lower bound value of 0.4 m be used in design for nailed decks. Although this may appear to be a very conservative approach, it should be noted that with deterioration, there is serious mechanical movement of the laminations which causes break-up of the wearing surface. Since it is not possible to specify a serviceability limitation which could be realistically linked with this condition, the conservative distribution for strength helps by lowering the loads the nails have to carry.

6.3/ Wood Sleeper With Plank Decking

This study performed the only known testing of this type of deck in Ontario. Due to the limited nature of these test results it is recommended that a conservative approach be maintained. An increase from 3 members to 4 members in the distribution represents a 33% increase and is the maximum recommended at this time. This assumes that the planks, as well as the sleepers and their connection to the girders, are maintained in good condition.

Dependent on the requirements of the MNR, who are the primary users of this bridge type, additional testing should be considered if further refinements are needed.

7/ References

[1] Csagoly, P.F. and Taylor, R.J., "Transverse Post-Tensioning of Longitudinally Laminated Timber Bridge Decks", Ministry of Transportation (MTO), RR220, Downsview, Ont., 1979.

[2] Taylor, R.J., Batchelor, B.deV. and Van Delan, K., "Prestressed Wood Bridges", CSCE International Conference on Short and Medium Span Bridges, Toronto, Ont., 1982. (MTO Reprint SRR-83-01).

[3] Bakht, B. "Load Distribution in Laminated Timber Bridge Decks", ASCE Journal of Structural Engineering, Vol. 114, No. 7, July, 1988.

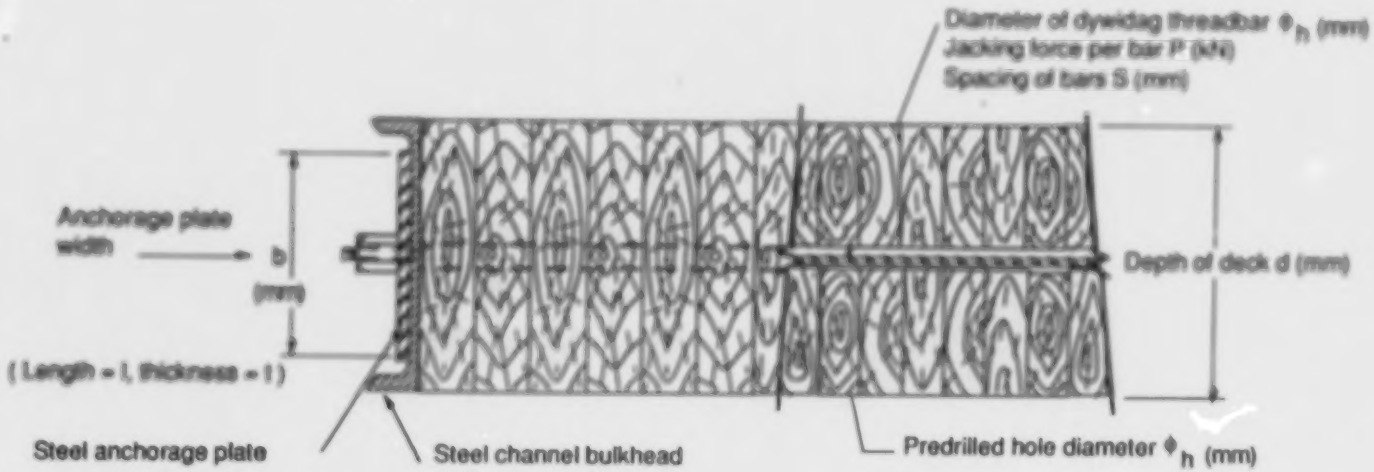
[4] Taylor, R.J. "Applications in Prestressed Wood", CSCE Second International Conference on Short and Medium Span Bridges, Ottawa, Ont., 1986. (MTO Reprint SRR-87-07).

[5] "Ontario Highway Bridge Design Code", MTO, Toronto, Ont., 1983 Edition.

[6] "Wood Handbook, Wood as an Engineering Material", US Forest Products Laboratory, USDA Agr. Hndbk 72, Rev., 1974.

Tables

Table 1/ Details of the Prestressed Wood Decks


Diameter of dywidag threadbar Φ_h (mm)
Jacking force per bar P (kN)
Spacing of bars S (mm)
Anchorage plate width
b (mm)
(Length = l, thickness = t)
Depth of deck d (mm)
Steel anchorage plate
Steel channel bulkhead
Predrilled hole diameter Φ_h (mm)

BRIDGE	d (mm)	Φ_h (mm)	Φ_b (mm)	S (mm)	P (kN)	Anchorage channel: Plate (mm) l x b x t	Anchorage channel: Bulkhead
Witch Doctor Lake Rd.	235	40	15.8	800	133	330 x 351 x 165	C230 x 30
Ragged Chutes	184	31.75	15.8	724	143	143 x 143 x 25	MC200 x 33.9
Victoria Island No. 1	235	31.75	15.8	610	139	191 x 191 x 25	MC250 x 61.2
Victoria Island No. 2 End span	184	31.75	15.8	737	144	143 x 143 x 25	MC200 x 33.9
Victoria Island No. 2 Main span	286	50	25	1448	417	298 x 238 x 32	MC310 x 74

Table 2/ Vehicle loading for Mississagi River Bridge

Load Level	Axle Loads (kN)					Total Vehicle Load (kN)
	Rear Tandem		Front Tandem		Steering	
	Axle No. 5	Axle No. 4	Axle No. 3	Axle No. 2	Axle No. 1	
1	89.9	89.9	88.5	88.5	47.9	405
2	134	134	135	135	52.7	591
3	158	158	157	157	54.9	685
4	181	181	180	180	57.2	779

Table 3/ Distribution widths derived from measured deflections

Bridge	Girder Spacing m	Deck Thickness mm	Average Load kN	D_t (m)			
				Mean	Standard Deviation	Minimum	Maximum
Prestressed							
Witchdoctor Lake	2.4	235	160	1.13	0.056	1.06	1.19
			243	1.13	0.068	1.03	1.17
			323	1.16	0.120	1.00	1.29
Victoria Island I	2.46	235	183	1.09	0.051	1.03	1.16
			272	1.10	0.042	1.06	1.15
			357	1.09	0.034	1.06	1.13
Victoria Island II End Spans	1.32	184	232	0.95	0.085	0.85	1.10
			321	0.98	0.102	0.85	1.11
			382	1.00	0.144	0.84	1.22
Victoria Island II Main Span	2.46	286	195	1.10	0.063	0.99	1.16
			283	1.10	0.059	0.99	1.15
			361	1.09	0.058	0.99	1.15
Ragged Chutes	1.42	184	184	0.96	0.049	0.87	1.01
			271	0.98	0.052	0.90	1.04
			357	0.98	0.054	0.91	1.05
Nailed							
Henderson Park	1.42	235	136	0.63	0.043	0.57	0.70
			180	0.61	0.045	0.55	0.68
			222	0.59	0.047	0.52	0.67
Wood Sleeper							
Mississagi River	1.40	184	63	1.14	0.078	1.01	1.20
			79	1.15	0.108	0.98	1.24
			90	1.20	0.135	0.98	1.35

**See Discussions in Section 5*

Table 4/ Proposed design distribution widths D_t for moment

Span (m)	Central Loading	Edge Loading*
0.91	0.68	0.62
1.22	0.84	0.78
1.83	1.15	1.01
3.05	1.68	1.48

**Edge Stiffening Assumed Equal to 0.2S Dy of Wood Deck*

Table 5/ Assumed vs. actual edge beam stiffness

Bridge	Stiffness of Edge Beam kN-M	
	Assumed (0.2S Dy)	Actual
Witch Doctor Lake	98,000	200,000
Victoria Island I	96,000	520,000
Victoria Island II, End Spans	30,000	211,000
Victoria Island II, Main Span	172,000	885,000
Ragged Chutes	62,000	211,000

Table 6/ Experimental distributions, Henderson Park Bridge

Age (years)	Dt		
	Mean (m)	Minimum (m)	Coefficient of variation %
1	0.66	0.47	9.7
7	0.59	0.45	7.9

Figures

Figure 1/ Chats Falls Access Bridge, Ottawa, Ontario

Figure 2/ Chats Falls, old wood ties & planking

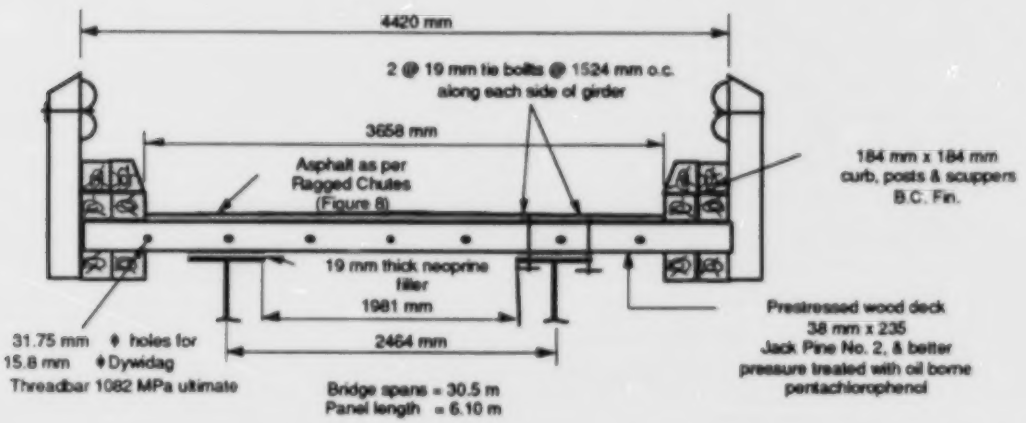


Figure 3/ Cross-section of Victoria Island Bridge I (Ontario Hydro)

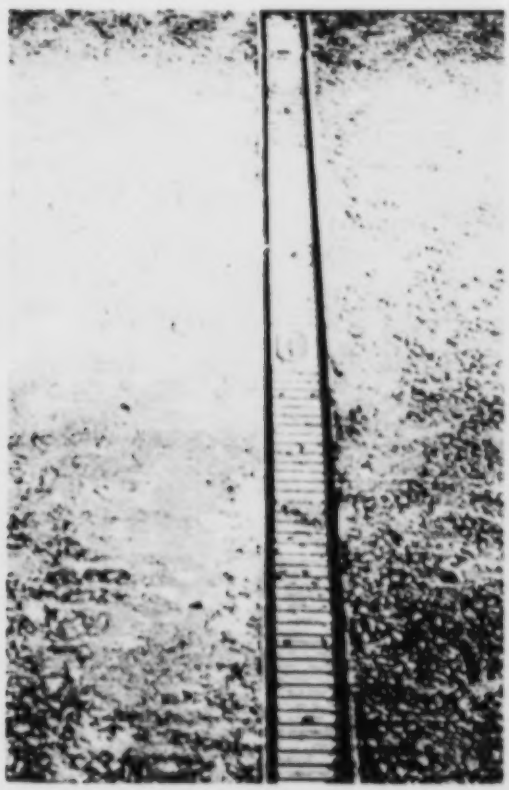


Figure 4/ Chats Falls, typical flex-seal joints

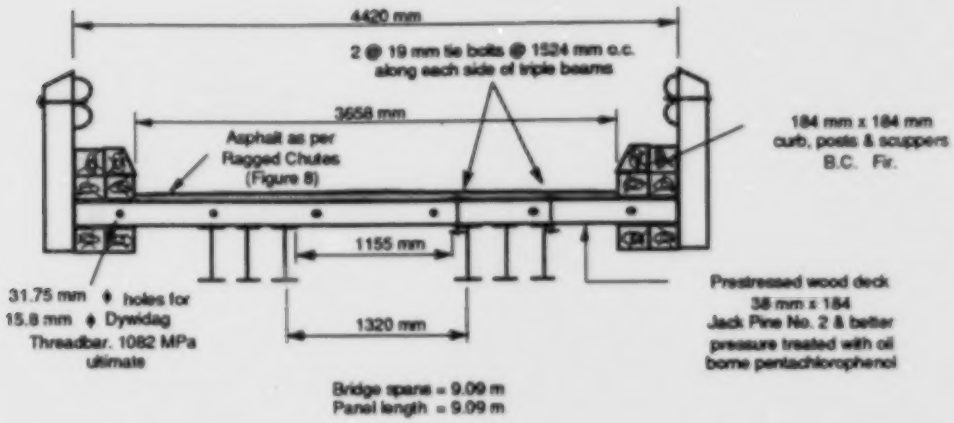


Figure 5/ Cross-section of Victoria Island Bridge II, end spans (Ontario Hydro)

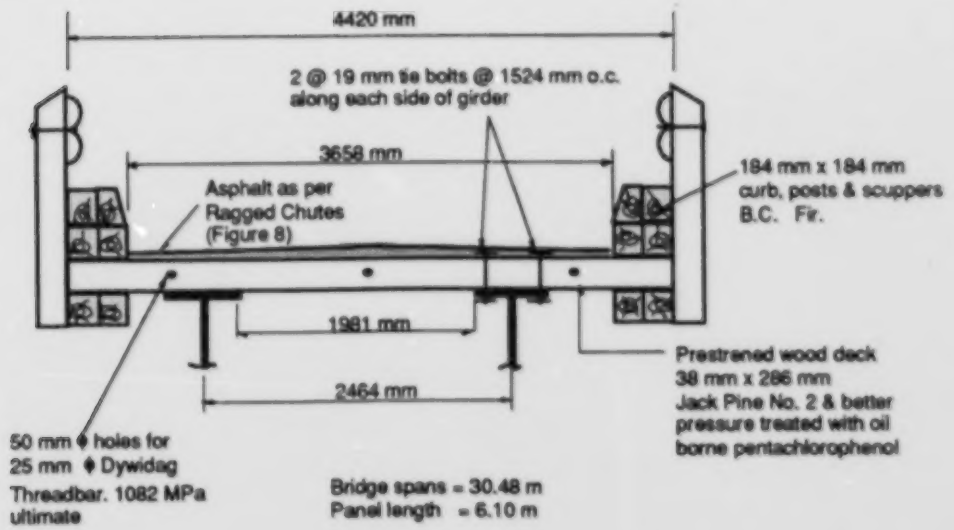


Figure 6/ Cross-section of Victoria Island Bridge II, main spans (Ontario Hydro)

Figure 7/ Ragged Chutes Bridges

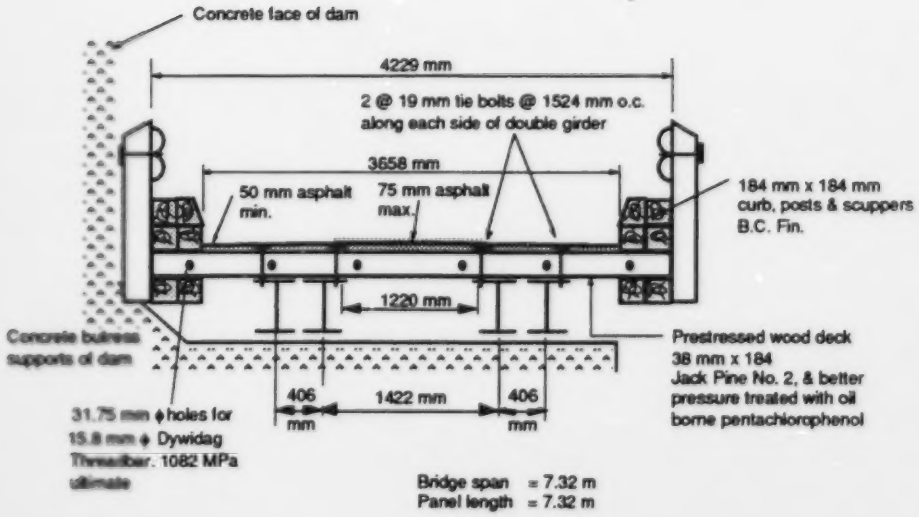


Figure 8/ Cross-section of Ragged Chutes Bridges (Ontario Hydro)

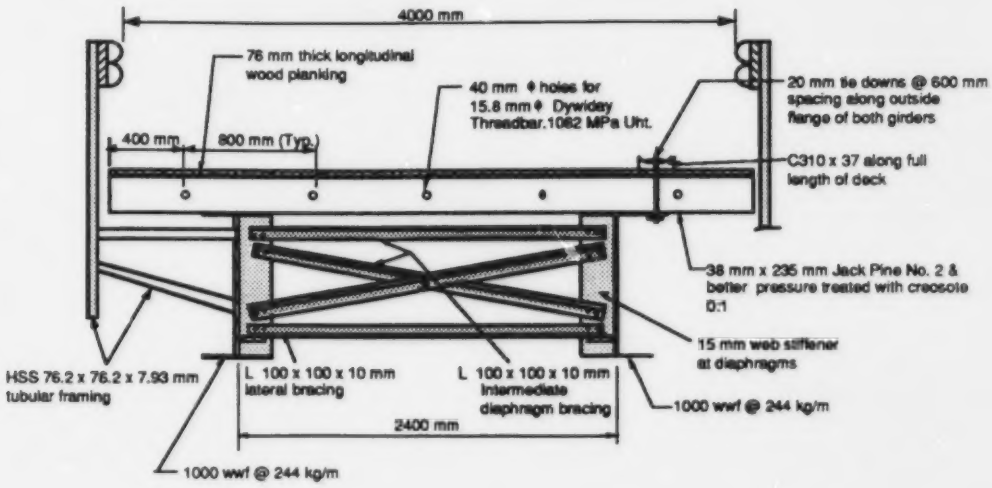

4000 mm
76 mm thick longitudinal wood planking
40 mm ϕ holes for 15.8 mm ϕ Dywidag Threadbar.1082 MPa Uht.
20 mm tie downs @ 600 mm spacing along outside flange of both girders
400 mm
800 mm (Typ.)
C310 x 37 along full length of deck
38 mm x 235 mm Jack Pine No. 2 & better pressure treated with creosote 0:1
15 mm web stiffener at diaphragms
HSS 76.2 x 76.2 x 7.93 mm tubular framing
L 100 x 100 x 10 mm lateral bracing
L 100 x 100 x 10 mm intermediate diaphragm bracing
1000 wwf @ 244 kg/m
2400 mm
1000 wwf @ 244 kg/m

Figure 9/ Cross-section of Witchdoctor Lake Road Bridge (MNR)

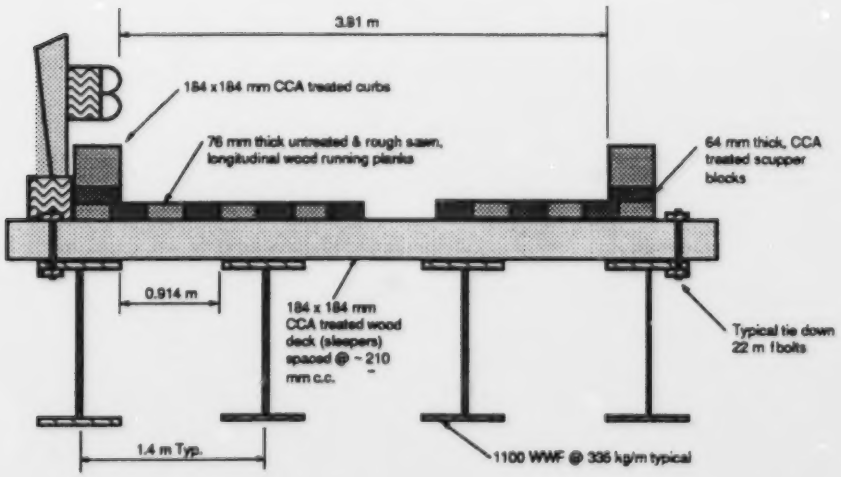

3.81 m
184 x184 mm CCA treated curbs
76 mm thick untreated & rough sawn, longitudinal wood running planks
64 mm thick, CCA treated scupper blocks
0.914 m
184 x 184 mm CCA treated wood deck (sleepers) spaced @ ~ 210 mm c.c.
Typical tie down 22 m f bolts
1.4 m Typ.
1100 WWF @ 336 kg/m typical

Figure 10/ Cross-section of Mississagi River Bridge (MNR)

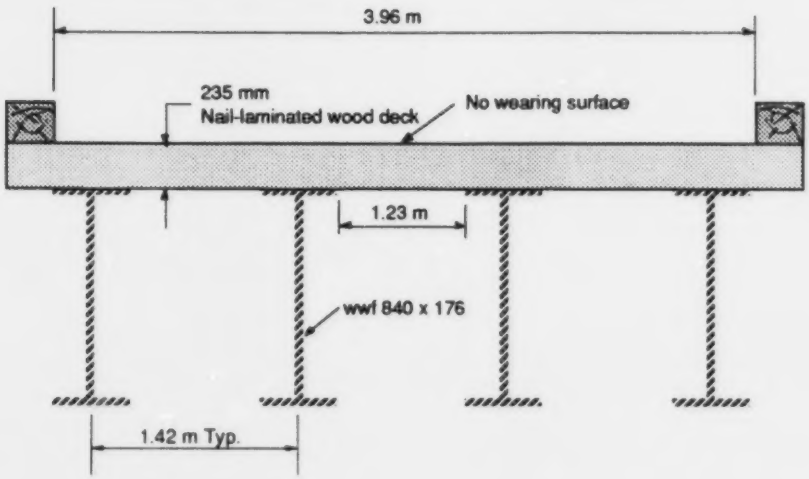


Figure 11/ Cross-section of Henderson Park Bridge MTO #10-113

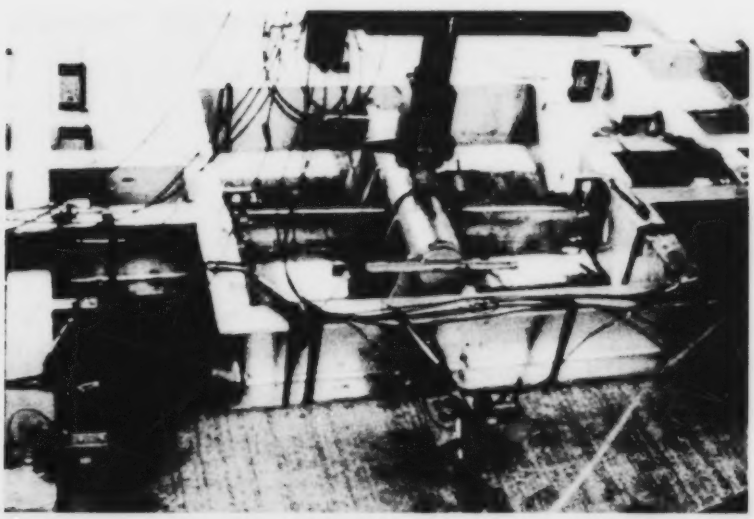


Figure 12/ MTO punching shear set-up

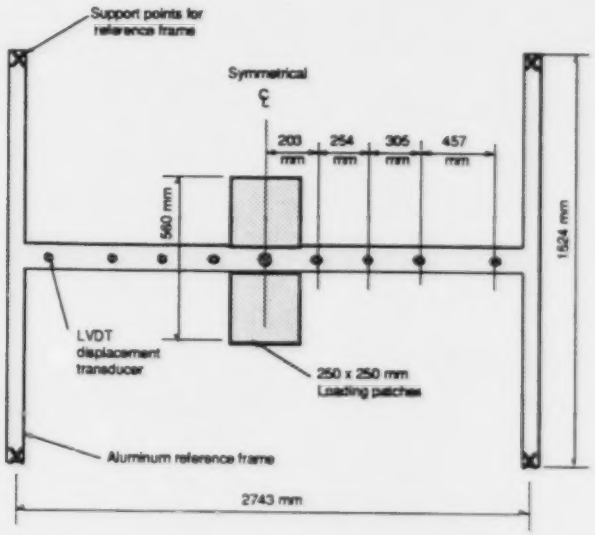


Figure 13/ Instrumentation and load location for Ontario Hydro and Henderson Park Bridge

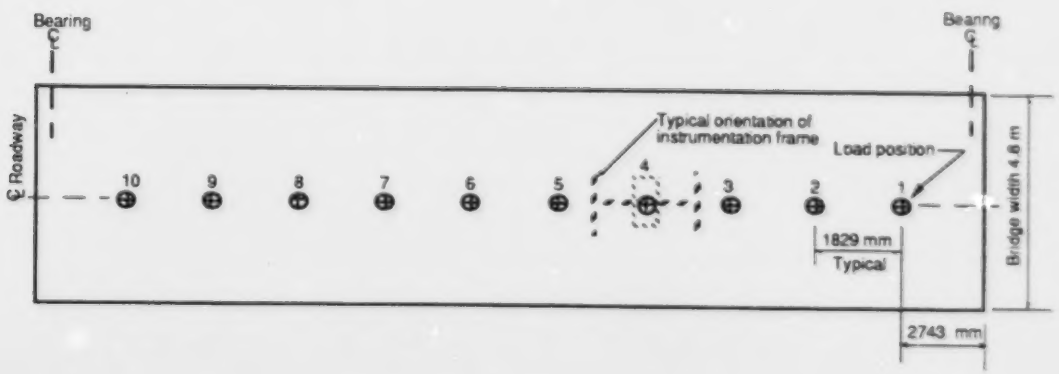


Figure 14/ Loading positions for Henderson Park Bridge

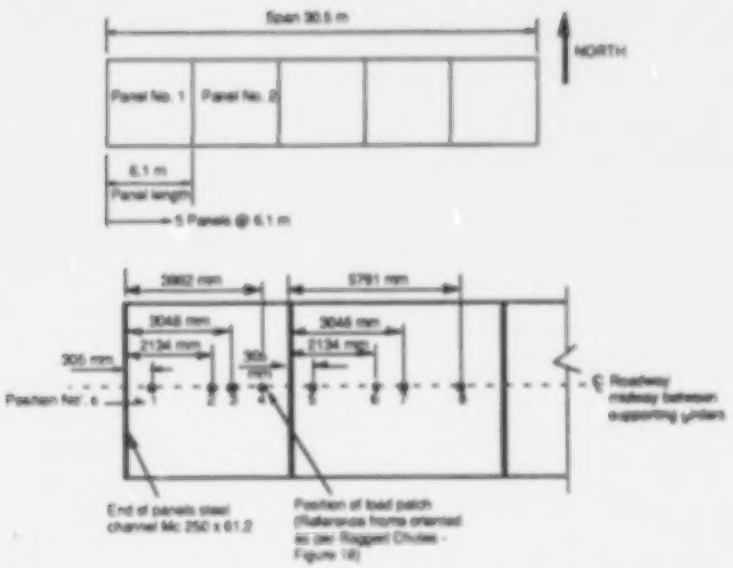


Figure 15/ Loading positions for Victoria Island Bridge I

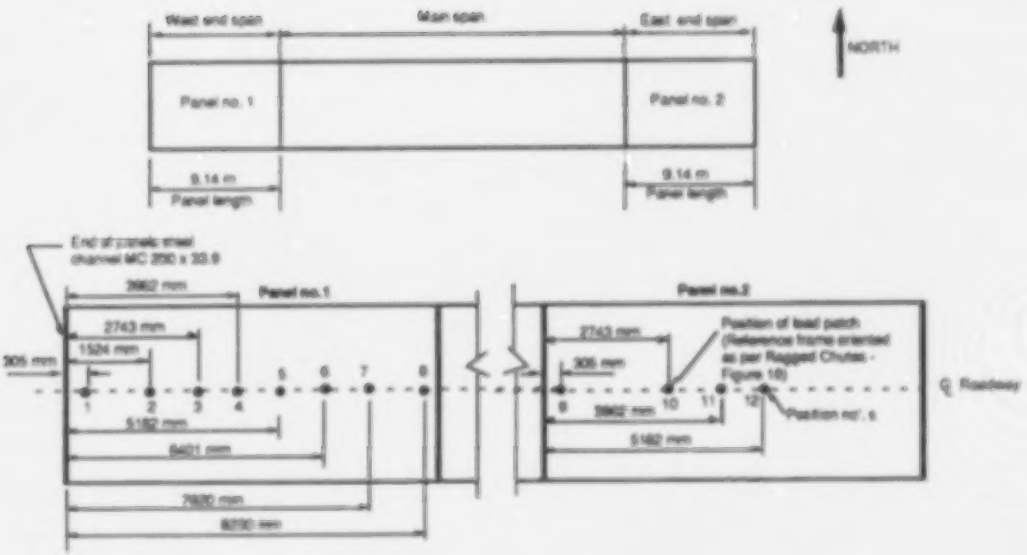


Figure 16/ Loading positions for Victoria Island Bridge II, end span

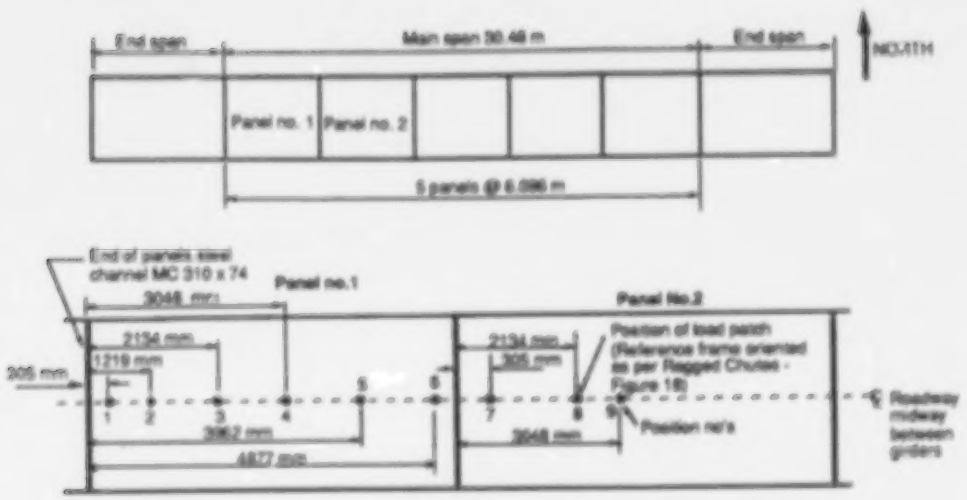


Figure 17/ Loading positions for Victoria Island Bridge II, main span

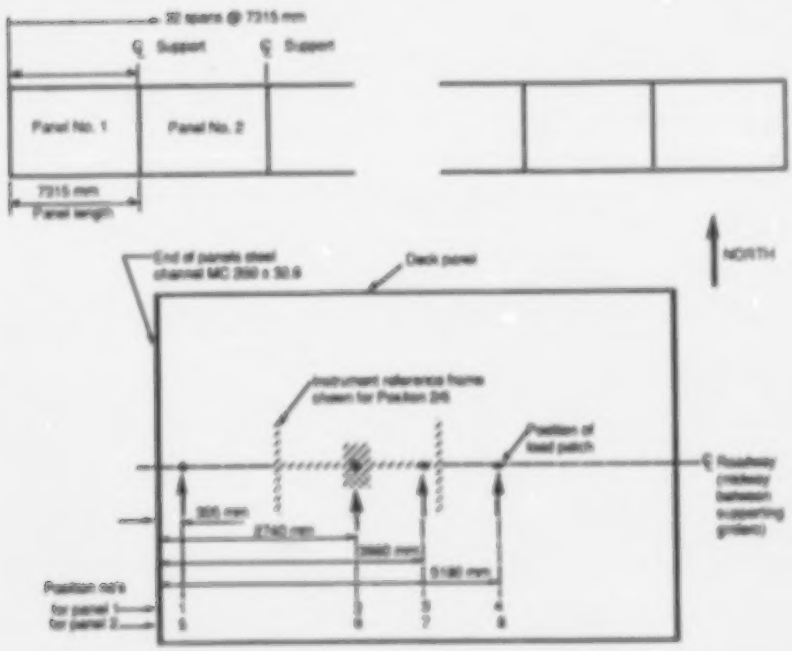


Figure 18/ Loading and instrument placement for Ragged Chutes Bridge

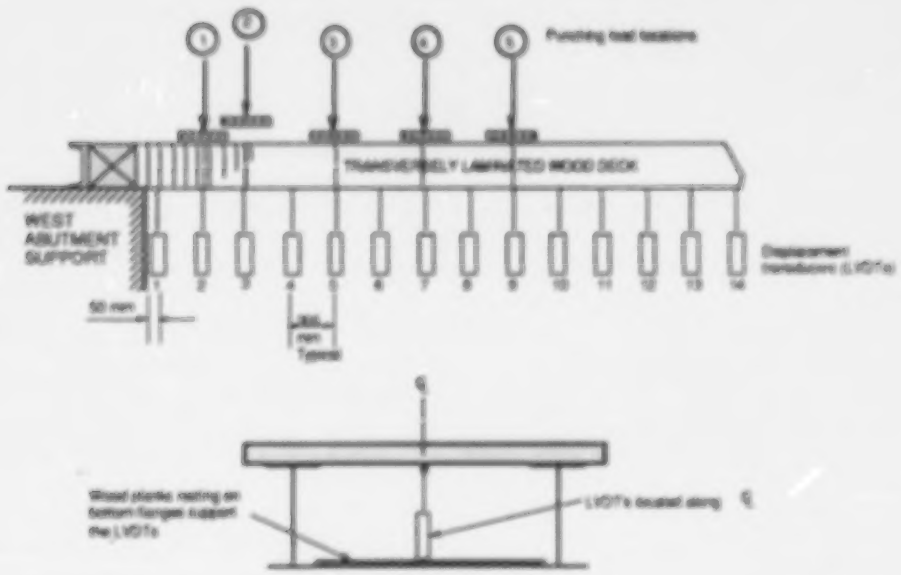


Figure 19/ Instrumentation and load positions for the Wichdoctor Lake Road Bridge

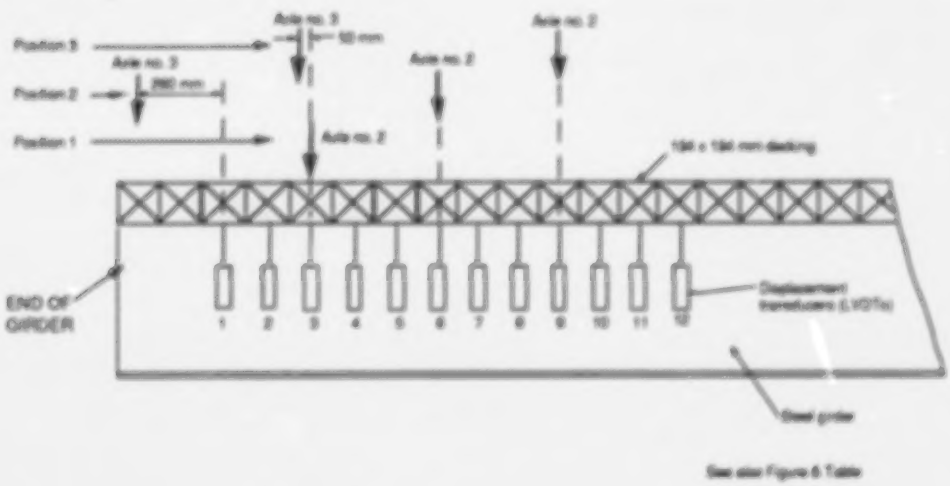


Figure 20/ Instrumentation and load positions for the Mississagi River Bridge

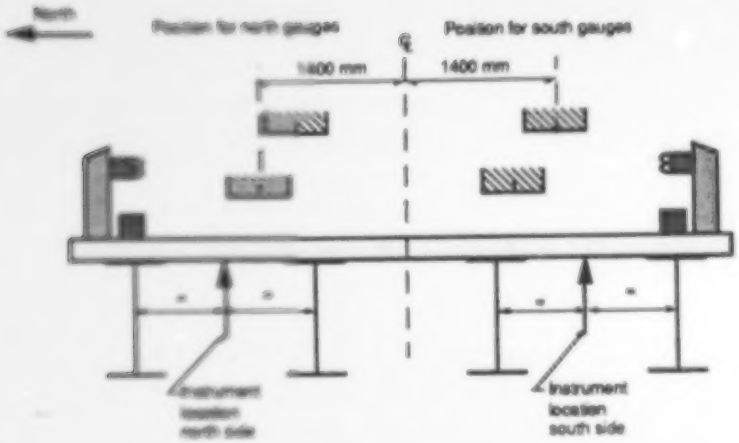


Figure 21/ Vehicle positions for the Mississagi River Bridge

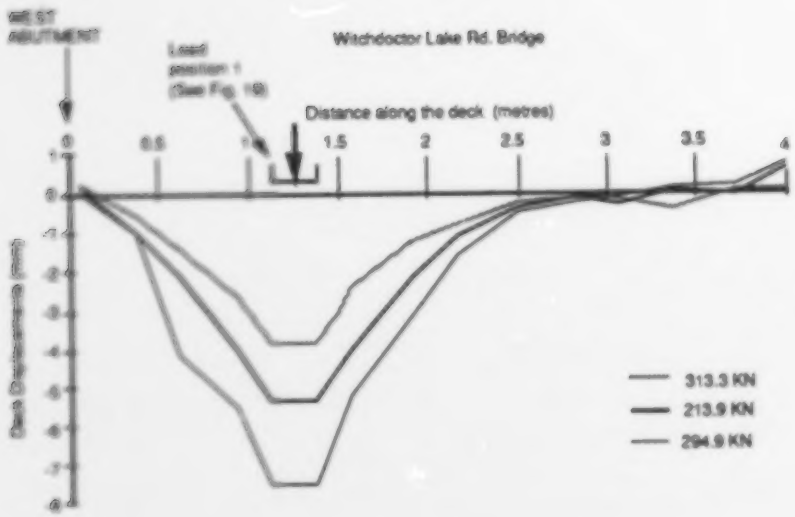


Figure 22/ Deck deflection profile, Witchdoctor Lake

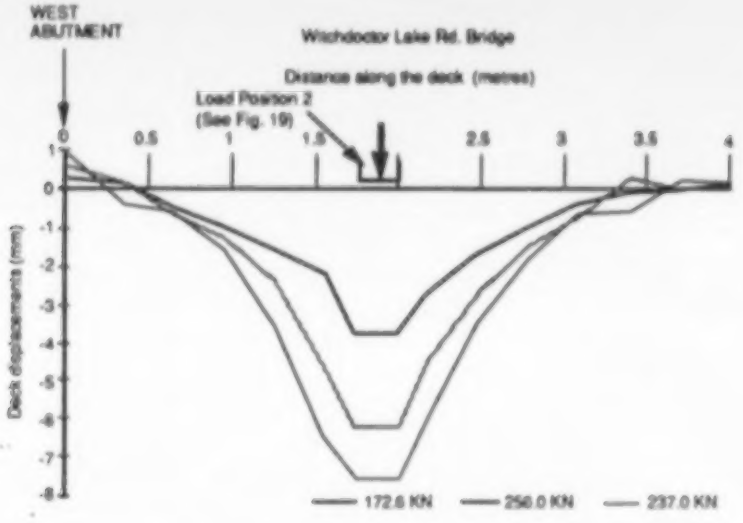


Figure 23/ Deck deflection profile, Witchdoctor Lake

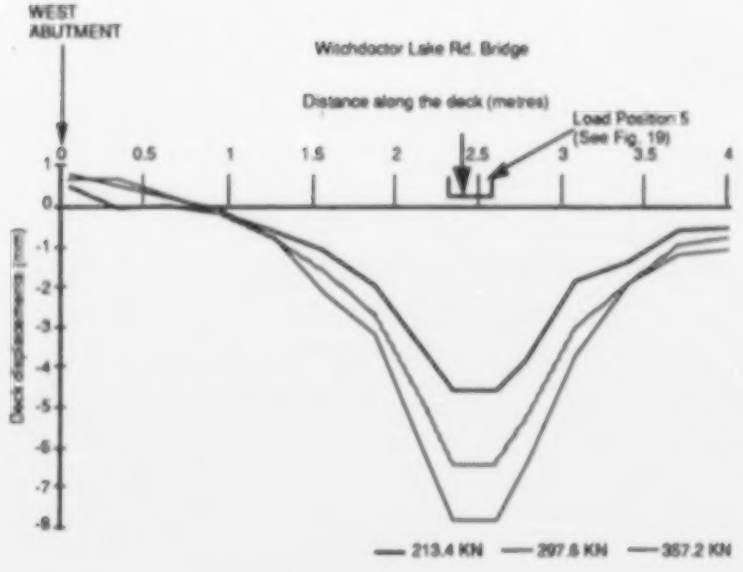


Figure 24/ Deck deflection profile, Witchdoctor Lake

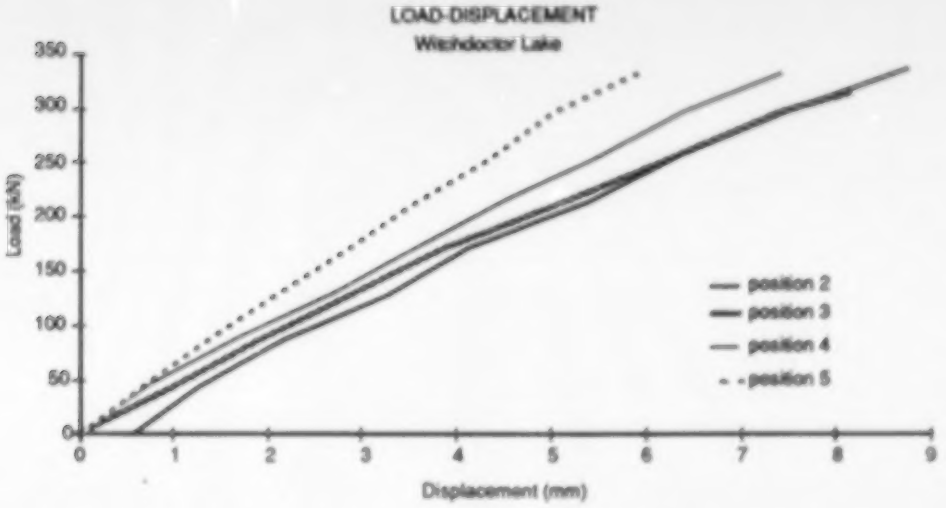


Figure 25/ Load-displacement, Witchdoctor Lake

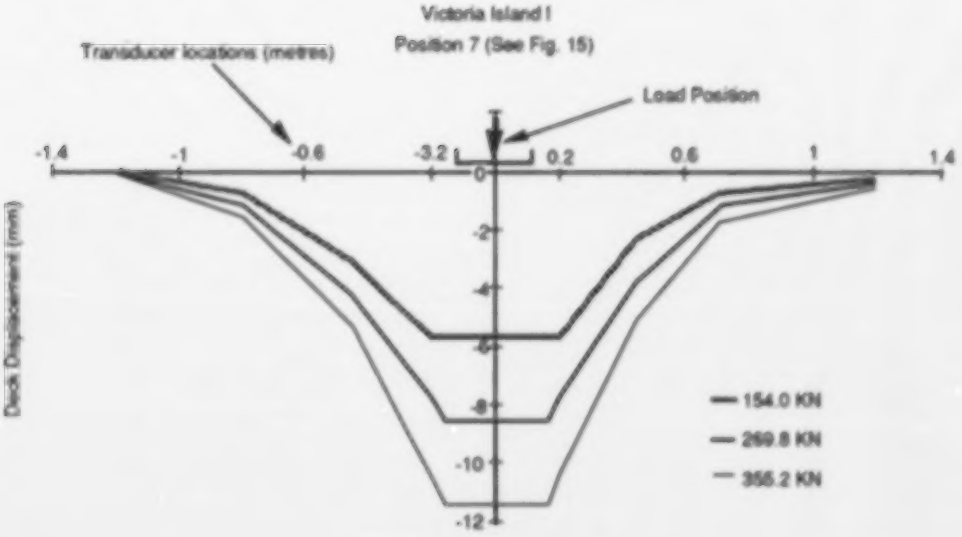


Figure 26/ Deck deflection profile, Victoria Island I

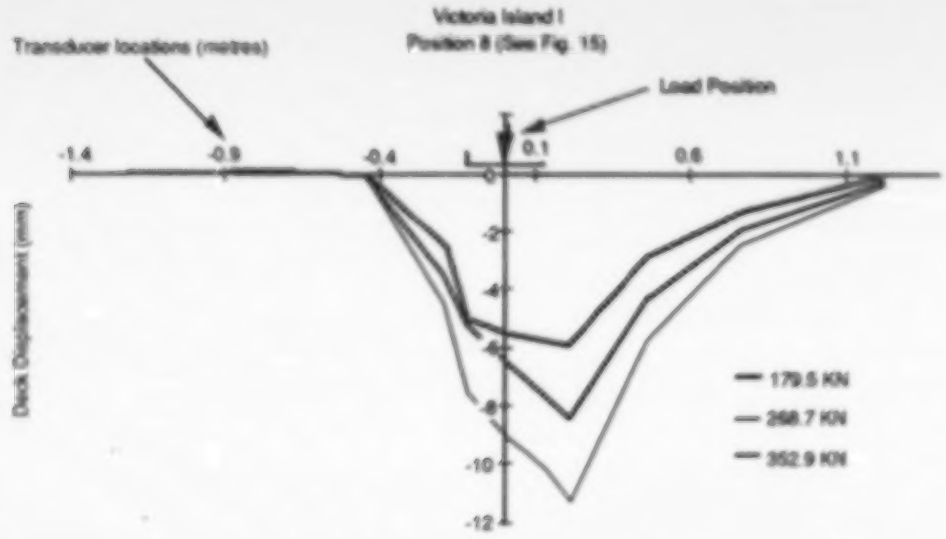


Figure 27/ Deck deflection profile, Victoria Island I

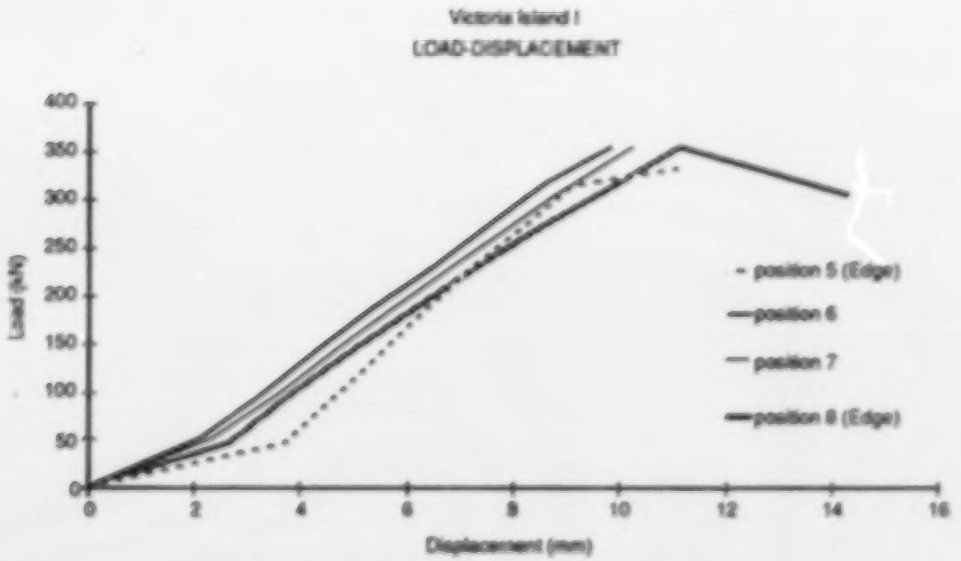


Figure 28/ Load-displacement, Victoria Island I

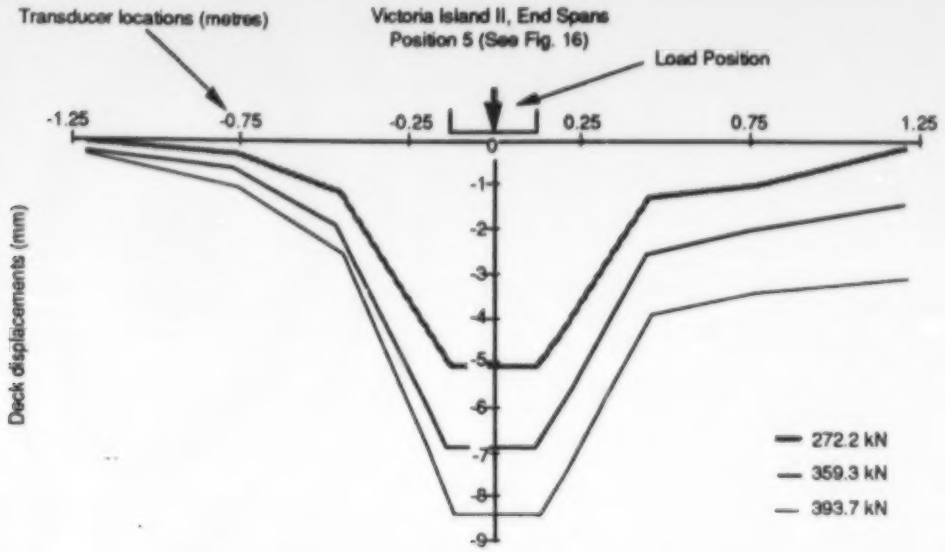


Figure 29/ Deck deflection profile, Victoria Island II Bridge, end spans

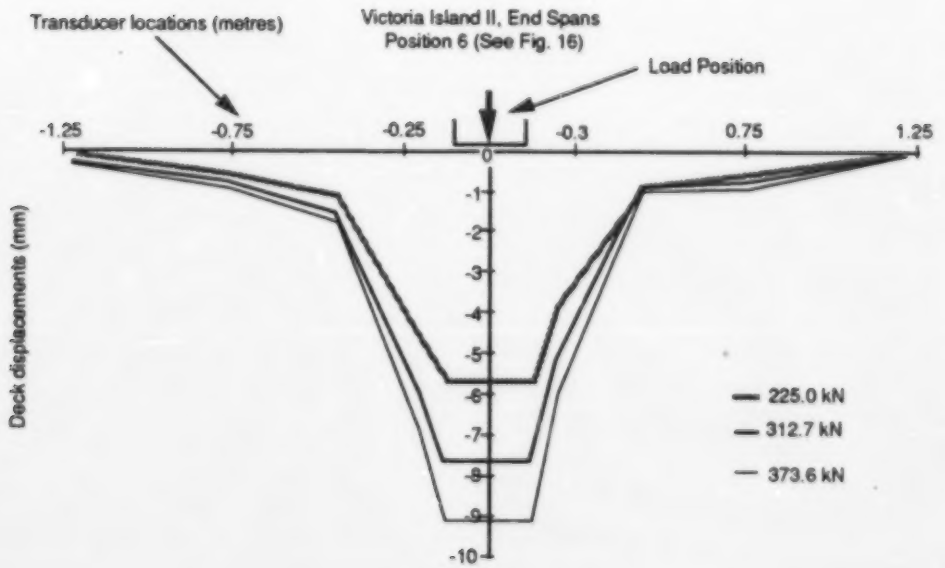


Figure 30/ Deck deflection profile, Victoria Island II Bridge, end spans

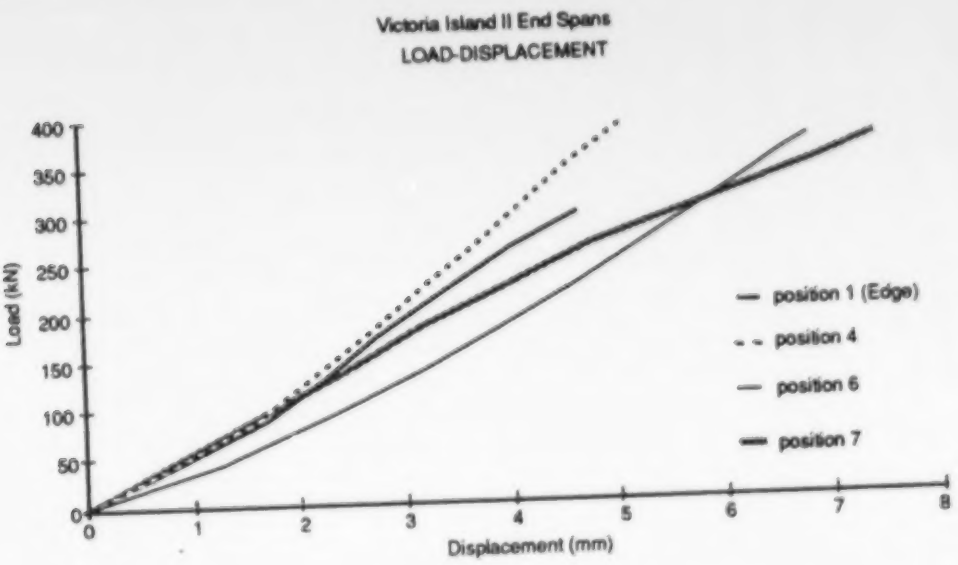


Figure 31/ Load-displacement, Victoria Island II Bridge, end spans

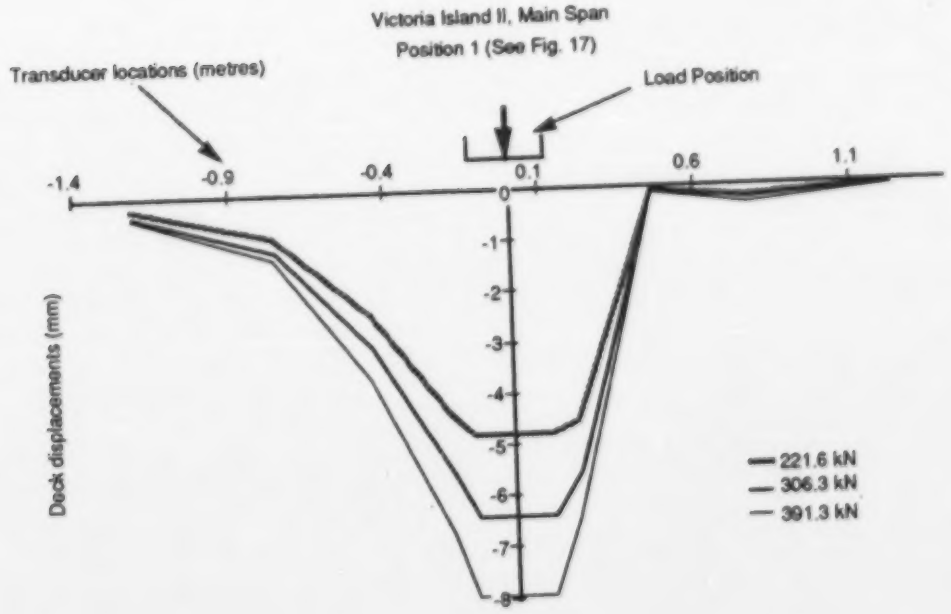


Figure 32/ Deck deflection profile, Victoria Island II Bridge, main span

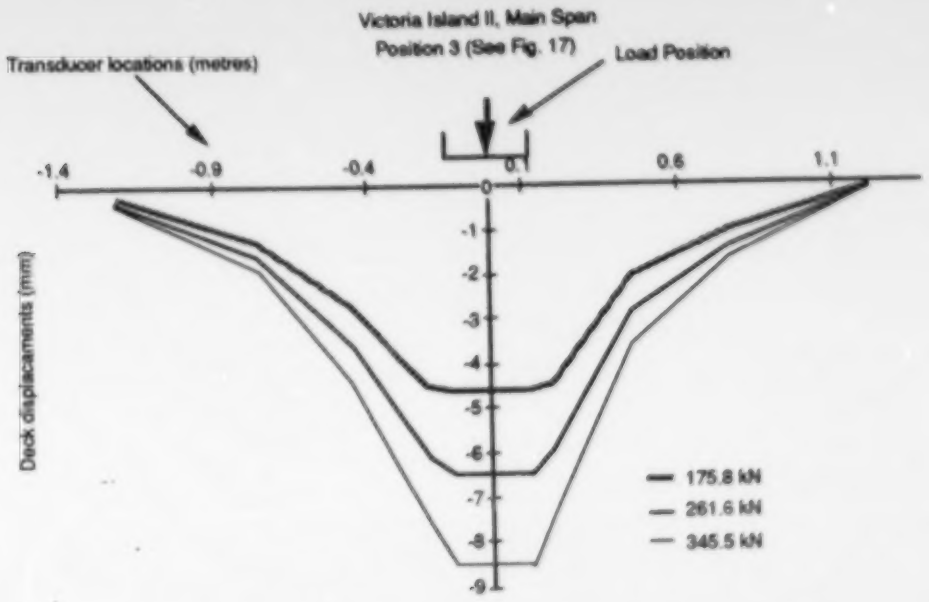


Figure 33/ Deck deflection profile, Victoria Island II Bridge, main span

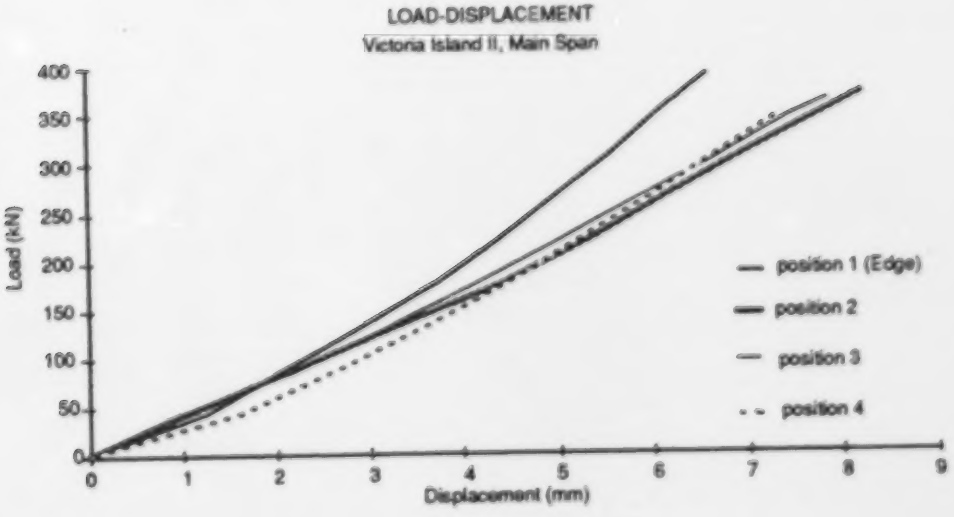


Figure 34/ Load-displacement, Victoria Island II Bridge, main span

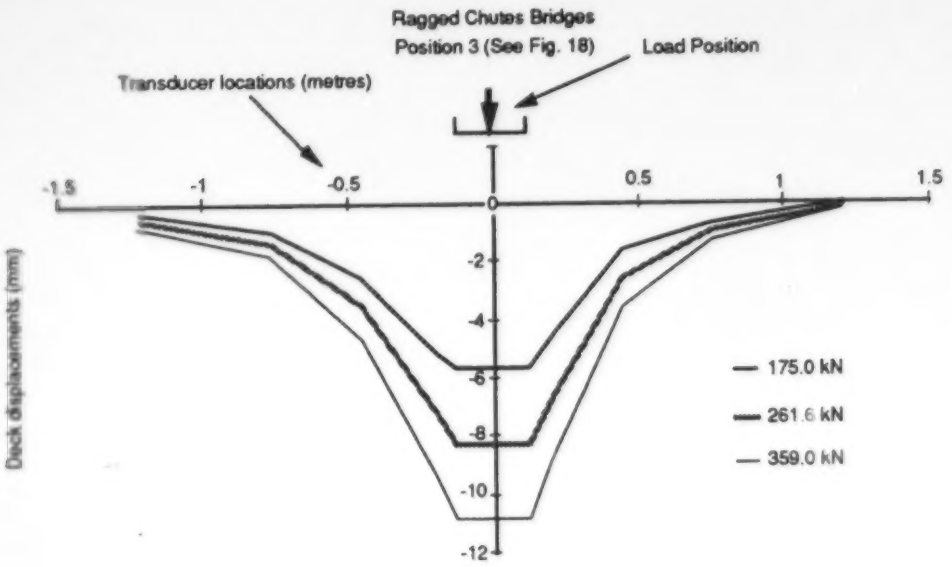


Figure 35/ Deck deflection profile, Ragged Chutes Bridges

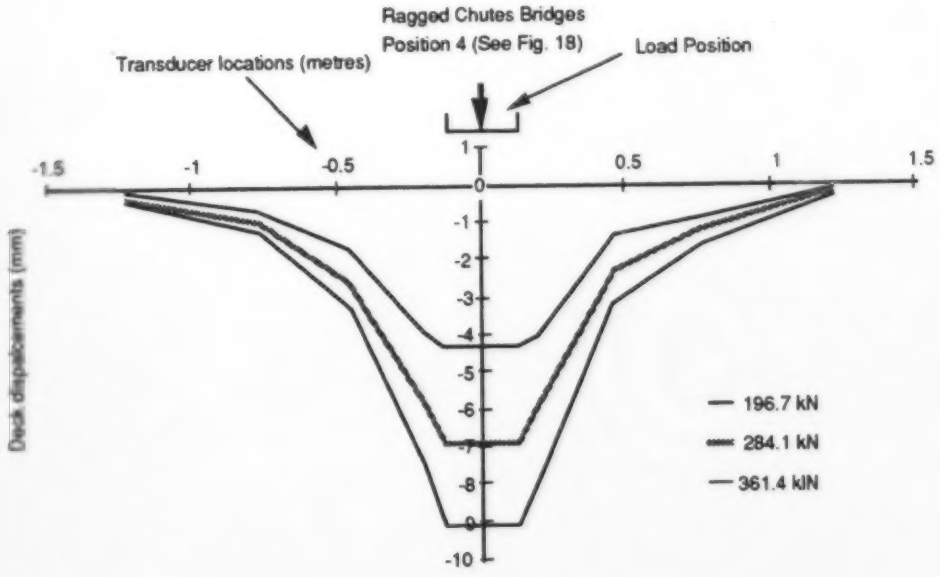


Figure 36/ Deck deflection profile, Ragged Chutes Bridges

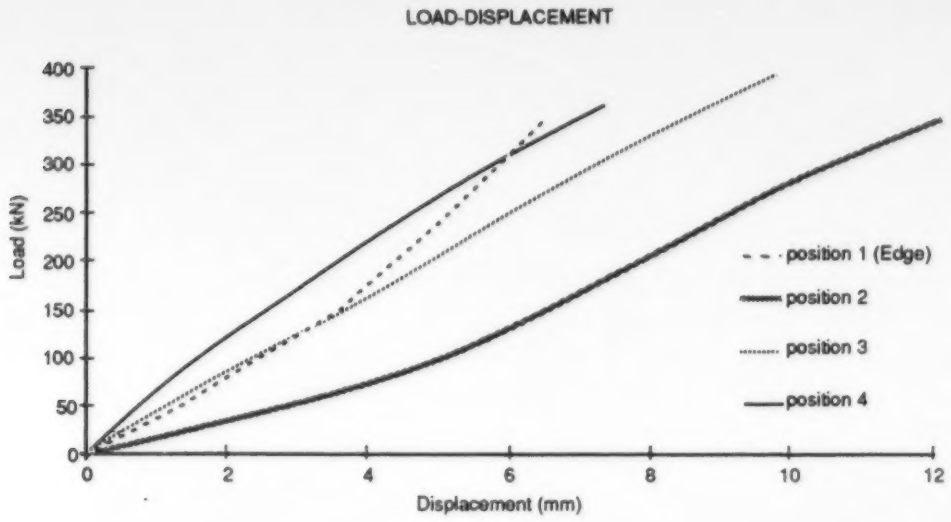


Figure 37/ Load-displacement, Ragged Chutes Bridges

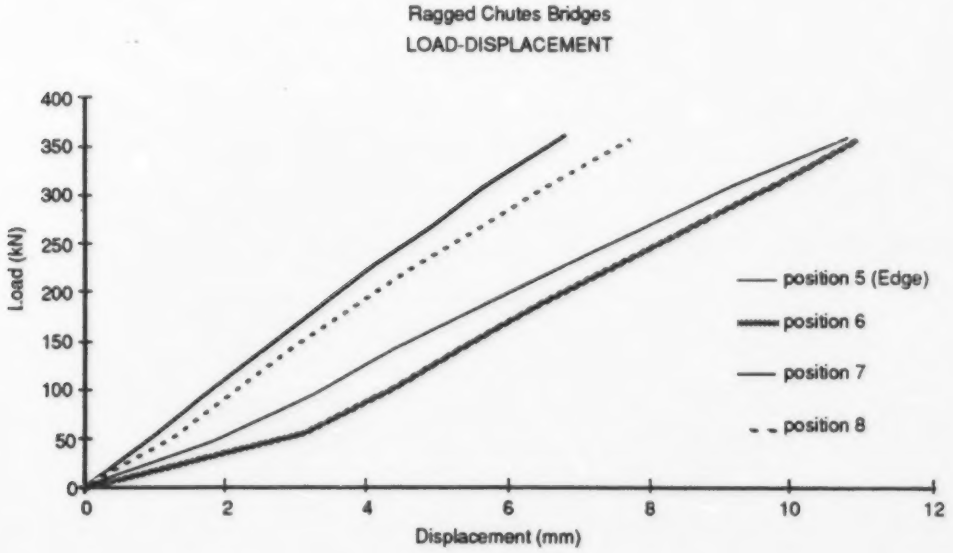


Figure 38/ Load-displacement, Ragged Chutes Bridges

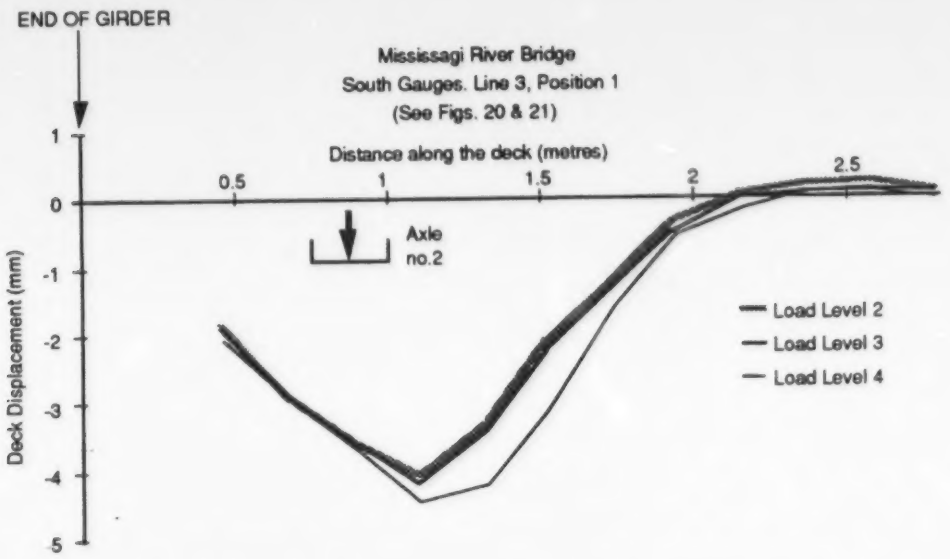


Figure 39/ Deck deflection profile, Mississagi River Bridge

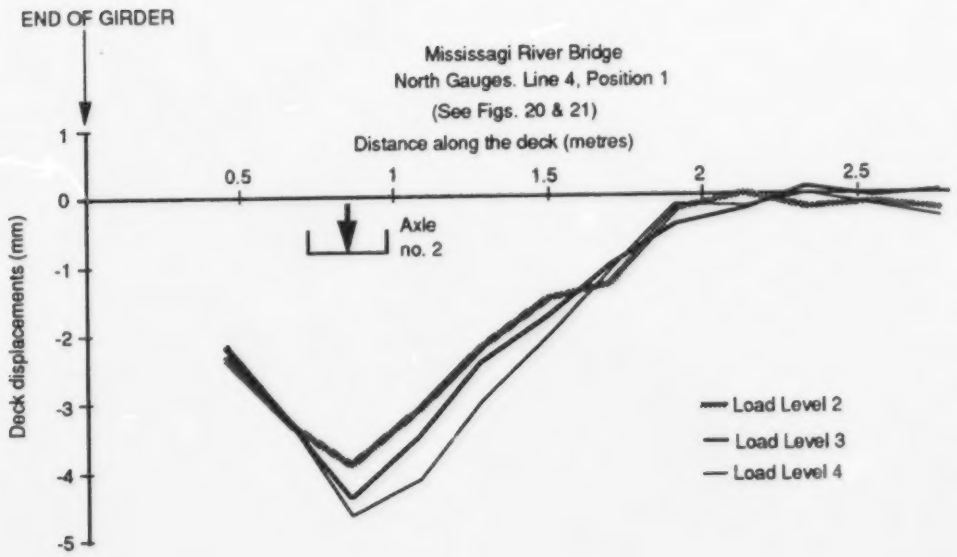


Figure 40/ Deck deflection profile, Mississagi River Bridge

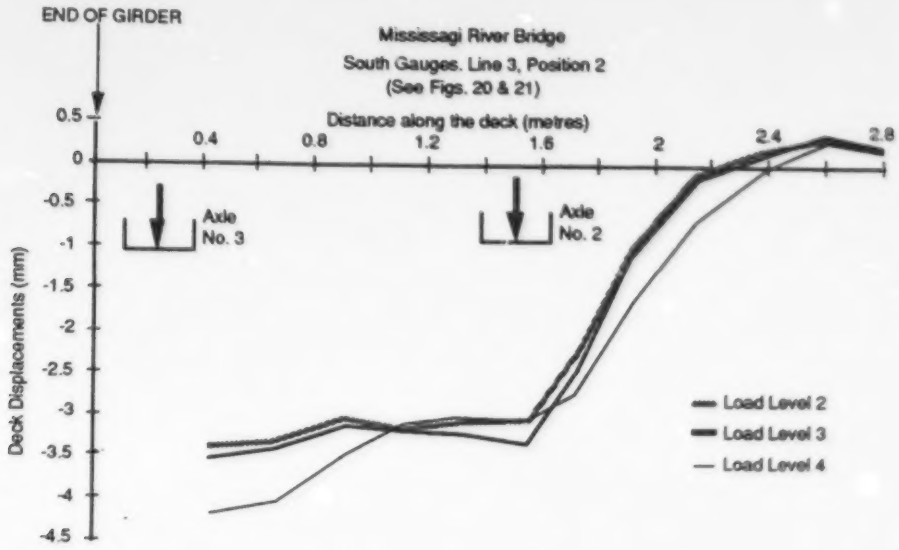


Figure 41/ Deck deflection profile, Mississagi River Bridge

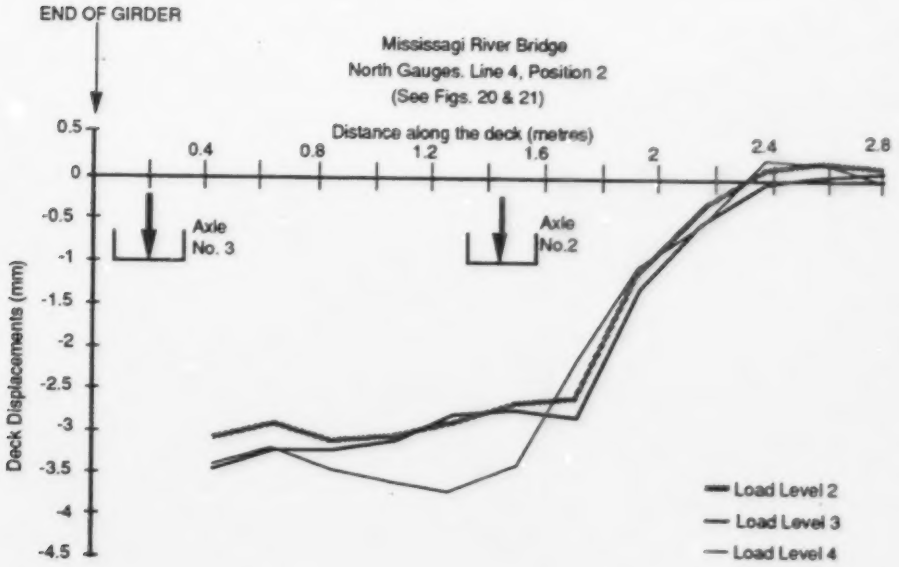


Figure 42/ Deck deflection profile, Mississagi River Bridge

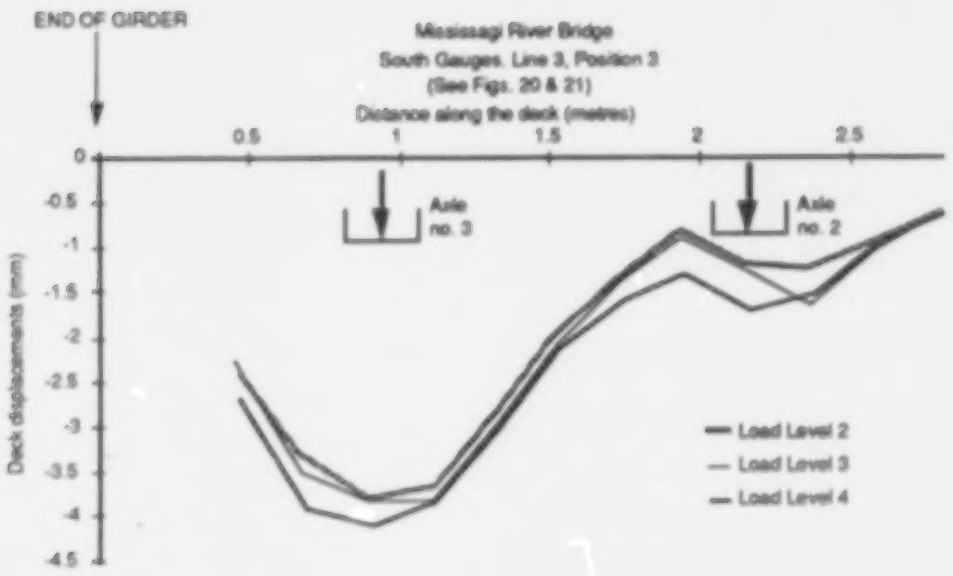


Figure 43/ Deck deflection profile, Mississagi River Bridge

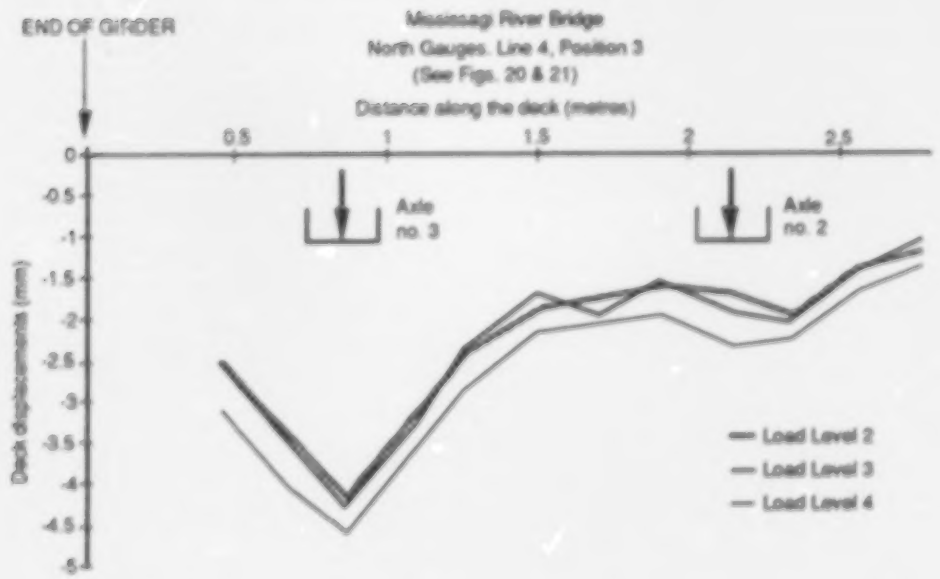


Figure 44/ Deck deflection profile, Mississagi River Bridge

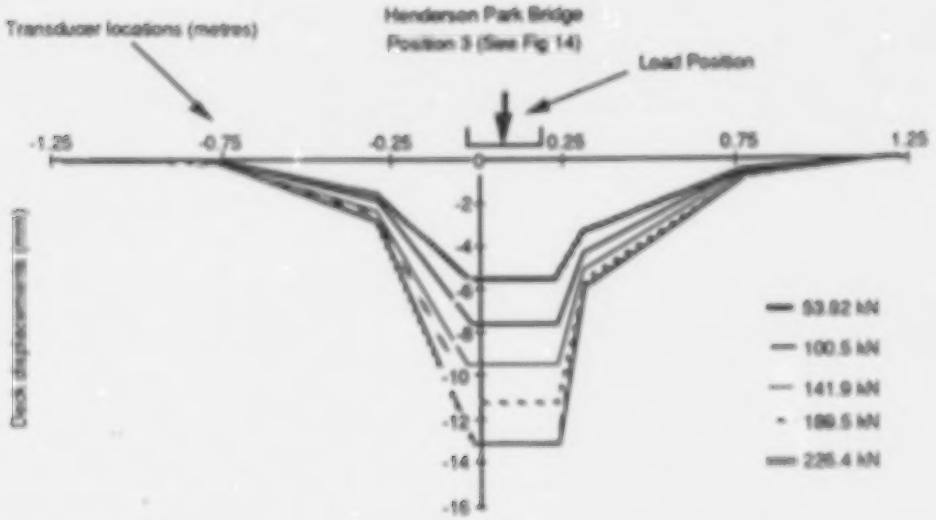


Figure 45/ Deck deflection profile, Henderson Park Bridge

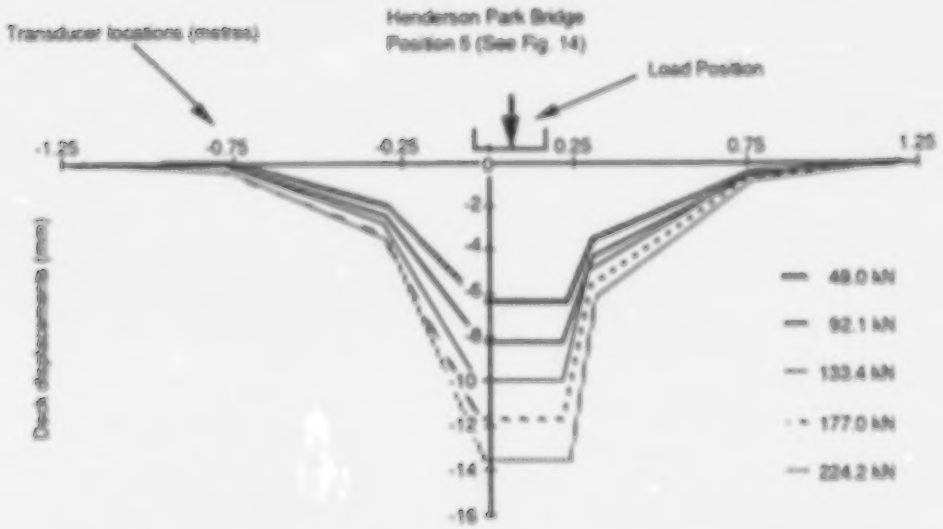


Figure 46/ Deck deflection profile, Henderson Park Bridge

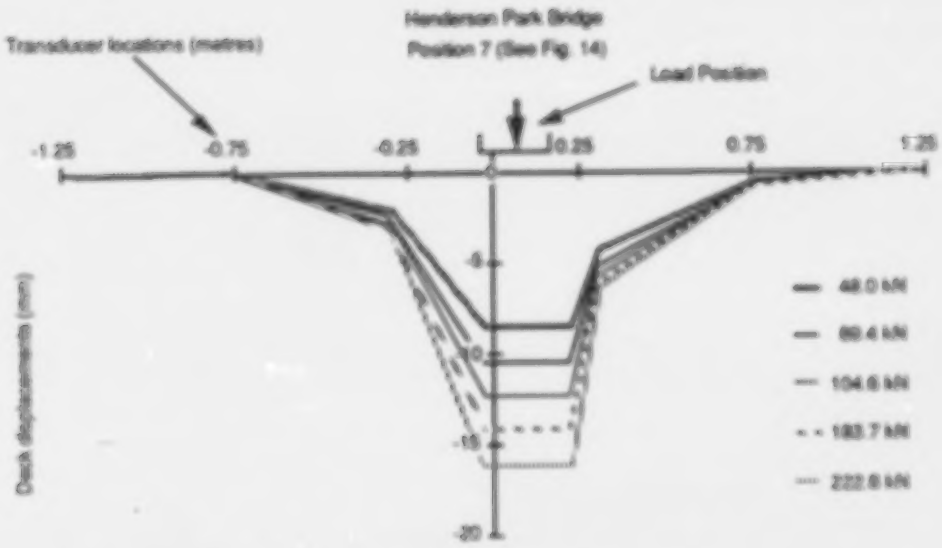


Figure 47/ Deck deflection profile, Henderson Park Bridge

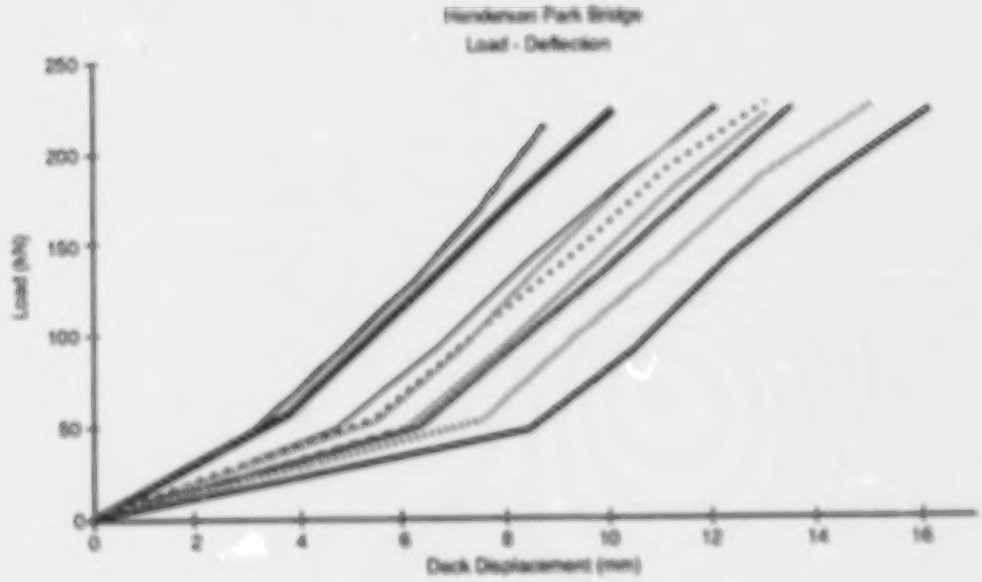


Figure 48/ Load-displacement, Henderson Park Bridge (7 years old)

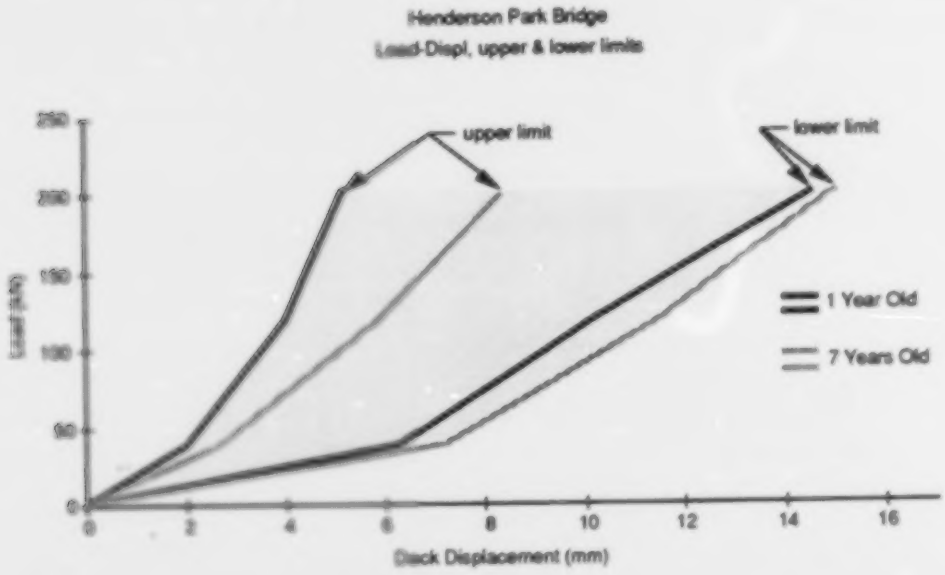


Figure 49/ Range of load-displacement curves, Henderson Park Bridge

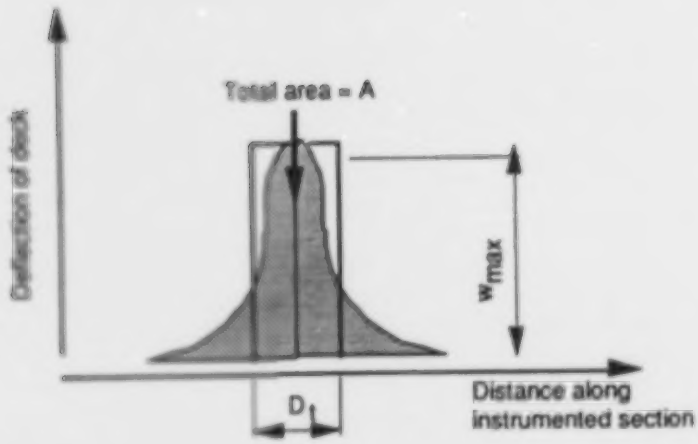


Figure 50/ Determination of D_t using measured deflections (Reference 3)

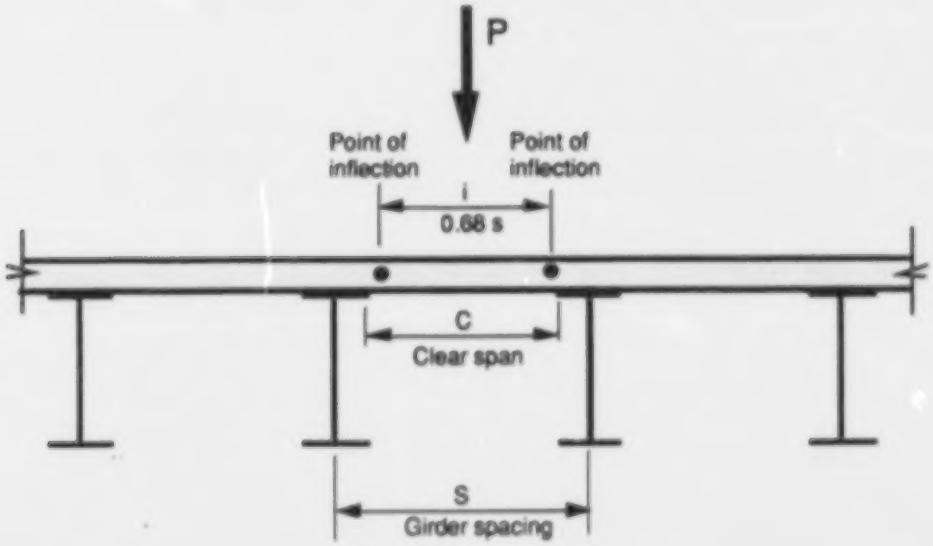


Figure 51/ Span assumption for continuous decks

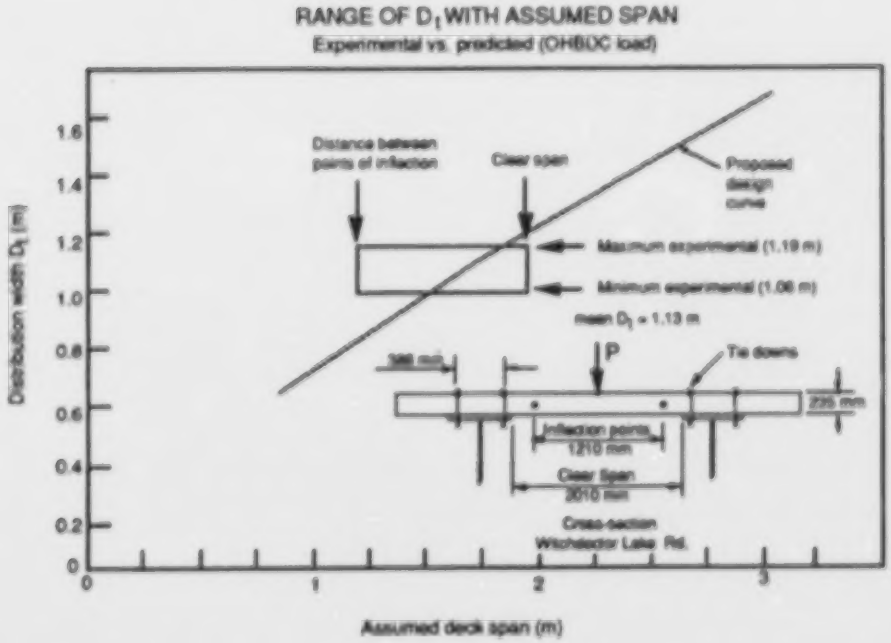


Figure 52/ Experimental vs. design distribution widths, Wichdoctor Lake Rd.

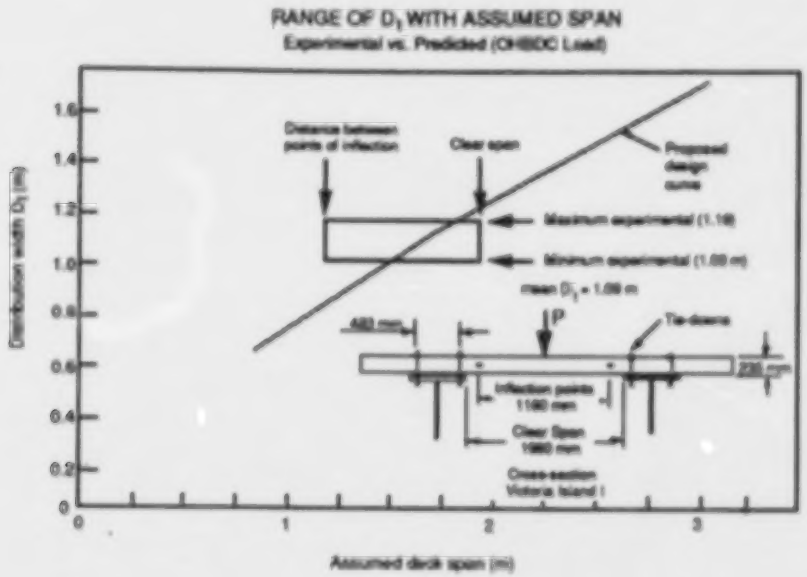


Figure 53/ Design distribution widths, Victoria Island I

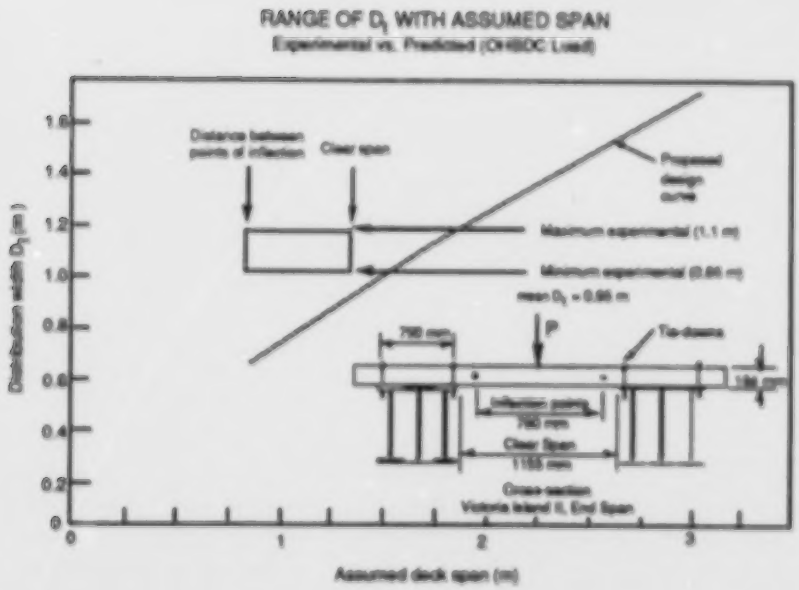


Figure 54/ Experimental vs. design distribution widths, Victoria Island II, end spans

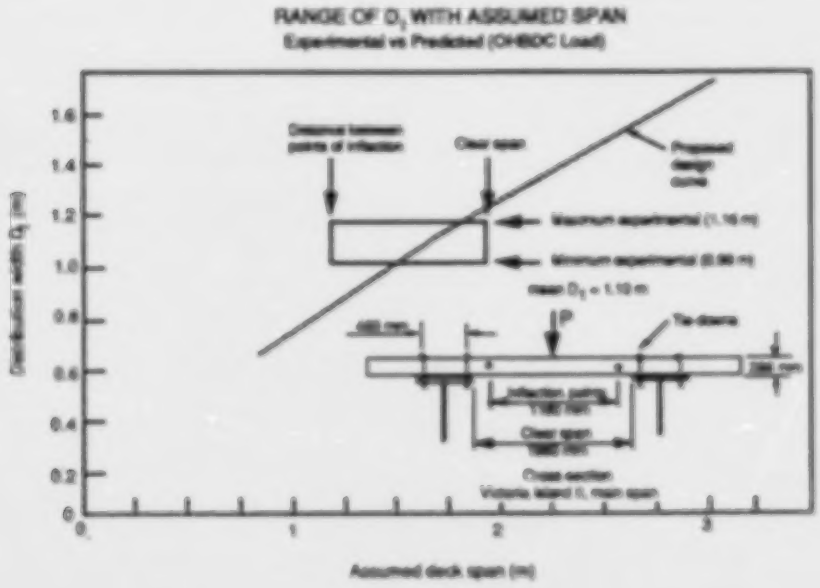


Figure 55/ Experimental vs. design distribution widths, Victoria Island II Bridge, main span

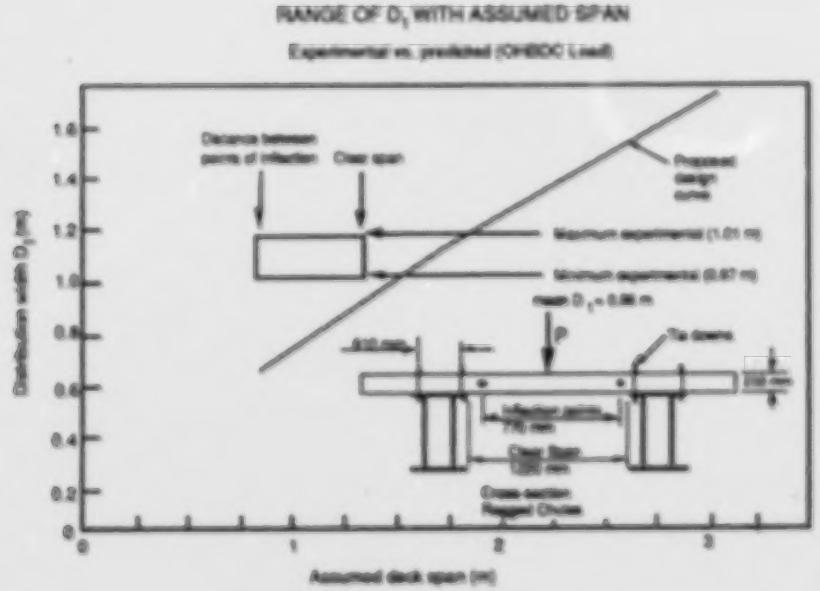


Figure 56/ Experimental vs. design distribution widths, Ragged Chutes Bridges

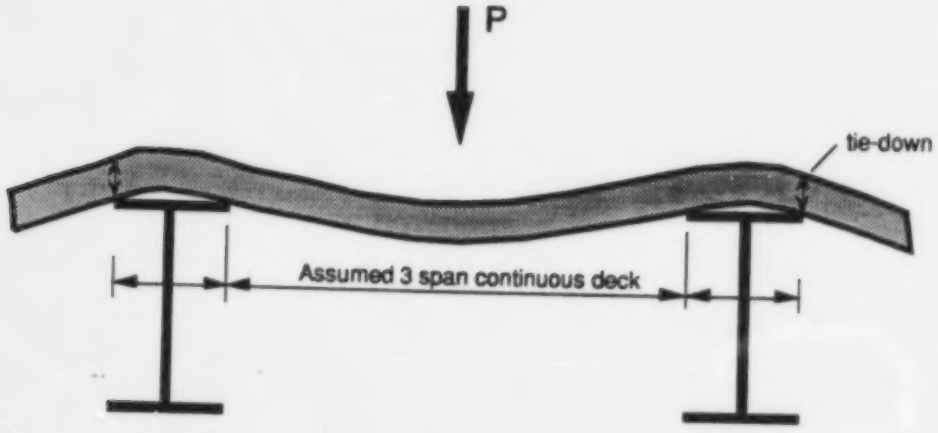


Figure 57/ Deck continuity due to tie-down

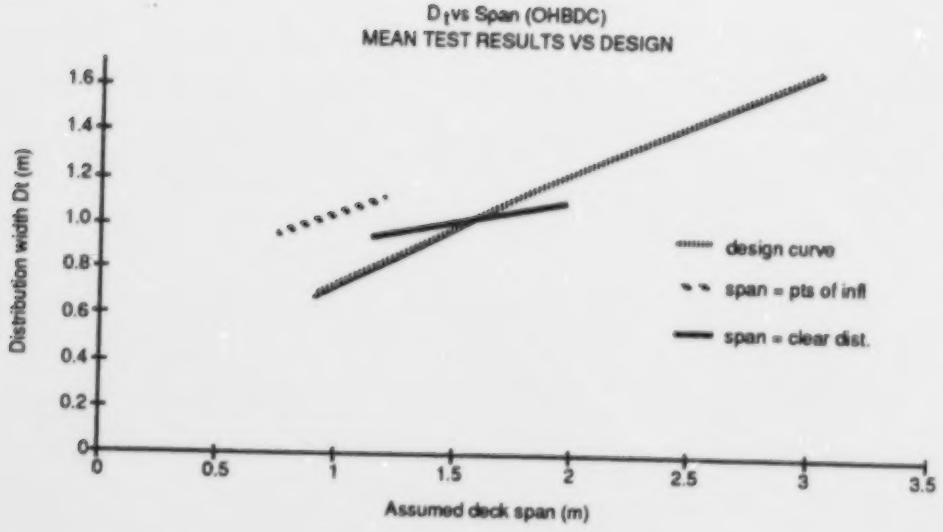


Figure 58/ Average experimental vs. design distribution widths, prestressed wood decks

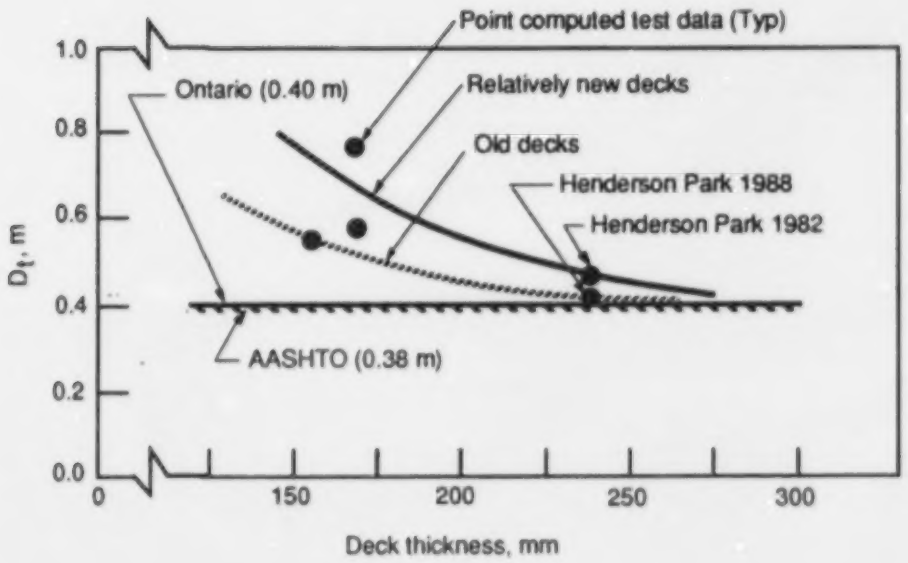


Figure 59/ Safest values of D_t for nailed decks computed from test data (Reference 3)